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HIGHWAY RESEARCH RECORD

Number 339

Symposium: Rapid Excavation

5 Reports

HIGHWAY RESEARCH BOARD

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Foreword

The papers in this RECORD comprise the formal presentations in a Symposium on Rapid Excavation Procedures. The current and future importance of rapid large-scale excavations is apparent from many sources. A sample of these includes the major underground construction involved in rapid transit systems in San Francisco, Chicago, and Washington; proposed use of tunnels for high-speed intercity travel in the northeast corridor; increasing use of underground openings for storage needs ranging from documents to liquefied natural gas; construction of highway tunnels of increasing length and depth; proposed creation of a sea-level Atlantic-Pacific canal; and the search for usable space in areas where the surface is already overcrowded. Although the applications described are diverse, they all involve removal of in situ earth and rock materials and support of the opening created as major components of the cost and time required for construction. Thus it is appropriate that these papers collectively should consider the present state of the art and projections for the foreseeable future in technologies applied to these tasks.

Day's paper describes the important progress in knowledge concerning the constructive use of large explosions along with an exciting view of the future prospects for nuclear construction technology.

Williamson's description of the relatively new tunnel-boring machines and his useful rules of thumb for preliminary investigation of their applicability will be of interest to all engineers concerned with tunneling. The current pace of technological change suggests that the 15-year lag between development of an idea and its acceptance in the field, as described by Williamson, is likely to be reduced in the future.

The lucid presentation of current tunnel support methods given by Deere et al. is an excellent review of existing techniques and portrays the need for innovative efforts in this important component of underground construction.

Irwin et al. emphasize clearly the importance of geologic investigation in selection of excavation methods and the enhanced significance of such investigations for successful rapid excavation techniques.

In drawing the four papers together, Lucke's summary indentifies important problem areas and at the same time indicates directions for the future in the important construction technologies discussed.

— W. H. Perloff

Contents

The Corps of Engineers Nuclear Construction Research Activities

WALTER C. DAY, U. S. Army Engineer Nuclear Cratering Group,
Lawrence Radiation Laboratory, Livermore, California

Program activities of the U. S. Army Corps of Engineers Nuclear Cratering Group include (a) cratering calibration of various geologic media and development of techniques to provide a desired crater geometry with chemical explosive detonations; (b) joint planning of and technical participation in Atomic Energy Commission nuclear excavation experiments; (c) development of data on the engineering properties of nuclear craters; (d) development of chemical and nuclear explosive construction technology for civil works; (e) engineering studies of nuclear construction feasibility; and (f) joint CE/AEC civil works nuclear construction experiments.

NCG has executed seven major chemical explosives cratering experiments to provide cratering calibration of dry alluvium, dry basalt, rhyolite, and water-saturated clay shale. Recently completed was a reservoir connection experiment at Fort Peck, Montana, and the cratering and safety calibration detonations for a small boat harbor excavation experiment in Kawaihae Bay on the Island of Hawaii. Nuclear crater engineering properties field investigations have recently been completed in which a trench was excavated through the lip of the crater and the material screened and weighed. Investigations are planned for some of the more recently executed nuclear cratering experiments. Estimates of true crater volume and radiation logs in drill holes on two other projects have been used as a basis for development of a technique for predicting the expected exposure dose rates in nuclear craters.

Four conceptual nuclear construction applications have been identified as having significant potential for accomplishment: (a) nuclear quarrying to produce rockfill or aggregate; (b) nuclear harbor construction; (c) nuclear ejecta dam construction; and (d) nuclear canal or roadbed cut excavation. The nuclear quarry has been identified as the most direct application of present technology. The large nuclear harbor at a remote site is one of the most attractive prospects for nuclear excavation.

•THE U. S. Army Corps of Engineers (CE) and the U. S. Atomic Energy Commission (AEC) have been engaged in a joint research program since 1962 to develop the basic technology necessary to use nuclear explosives in conjunction with the construction of large-scale civil engineering projects.

Under the agreement for the joint research program, the AEC is primarily responsible for nuclear explosive development, execution of nuclear cratering experiments, and development of methods for predicting the size and shape of nuclear craters. The major AEC effort in this research program is accomplished by the Lawrence Radiation Laboratory in Livermore, California. The Corps of Engineers is primarily responsible for execution of corollary chemical explosive cratering experiments, technical participation in and assistance in the planning of the AEC's nuclear cratering experiments, and development of the requisite engineering and construction data to be used as the

Paper sponsored by Committee on Soil and Rock Properties and presented at the 49th Annual Meeting.

basis for using nuclear explosives for construction purposes. The U. S. Army Engineer Nuclear Cratering Group (NCG) is located at the Lawrence Radiation Laboratory and is responsible for technical program direction effort of the Corps.

In addition to this joint research effort with the AEC in nuclear explosive construction, the mission of NCG has recently been expanded to include research aimed at the use of chemical explosives for projects of intermediate size. This mission has grown out of the experience gained in our chemical explosive cratering experiments. The chemical explosive experiments currently being accomplished by NCG are intended to provide experience and data useful in both the joint program with the AEC and NCG's expanded mission in the use of chemical explosives in construction.

The basic concept of nuclear construction (1) involves the subsurface detonation of nuclear explosives either to break up and eject large quantities of rock and/or soil and by so doing produce excavations that may be used as engineering structures, such as channels, harbors, dams, or spillways, or to simply break up rock to produce a quarry. The primary advantage in using nuclear explosive methods rather than conventional construction methods is economy. The nuclear cratering experience to date indicates that there is a significant potential for using nuclear explosives to accomplish large-scale construction projects at considerable savings in cost and time.

The use of nuclear explosives for construction involves more than merely producing craters or mounds of rock. One must be able to predict the geometry of the crater or, better still, produce a desired geometry to fit a specific application. In addition, one must know the extent of the disturbance to the media that has occurred immediately adjacent to the crater for those applications involving use of the crater as an engineering structure. Also, it is necessary to have detailed knowledge of nuclear explosive characteristics and handling and emplacement requirements as well as an understanding of the extent and safety implications of airblast, ground shock, and residual radioactivity effects that occur as a result of nuclear cratering detonations. The objective of the nuclear excavation research program is to develop the technology required to address these areas of interest.

The NCG program activities include (a) cratering calibration of various geologic media and development of techniques designed to provide a desired crater geometry with chemical explosive detonations; (b) joint planning of and technical participation in AEC nuclear excavation experiments; (c) development of data on the engineering properties of nuclear craters; (d) development of civil works chemical and nuclear explosive construction technology; (e) accomplishment of engineering studies of nuclear construction feasibility; and (f) execution of joint CE/AEC civil works nuclear construction experiments.

This paper summarizes the scope of NCG research activities and discusses the results of the most recent programs.

CRATER GEOMETRY

There are basically two approaches that have been developed to date for predicting crater dimensions. One approach involves computer calculations of the mound and cavity growth used in conjunction with a freefall, throwout model that gives a reasonable estimate of the crater radius and ejecta boundary. The second approach involves empirical scaling relationships.

The Plowshare Division of the Lawrence Radiation Laboratory has developed the SOC (spherical, one-dimensional) and TENSOR (cylindrical, two-dimensional) computer codes that numerically describe the propagation of a stress wave of arbitrary amplitude through a medium (2, 3). These codes are Lagrangian finite-difference approximations of the momentum equations that describe the behavior of a medium subjected to a stress tensor in one (SOC) and two (TENSOR) dimensions. The code calculations handle both the initial shock wave, which creates spall velocities, and the gas acceleration phase. The end product of the TENSOR code calculations is a chronological history of the cavity and mound growth resulting from an underground explosive detonation. The code calculation runs until the particle velocities no longer increase significantly from cycle to cycle. At this point, a freefall, throwout model

calculation is used to determine the mode of deposition of that material which has been given sufficient velocity to pass the original ground surface. The ballistic trajectory of any given mass determines its final position on the surface. The throwout model calculation permits one to estimate crater radius and the maximum range to which significant material is thrown by the detonation. An estimate of the crater depth may also be made by considering the stability of the cavity walls and the bulking characteristics of the material that falls back into the crater opening.

The second crater geometry prediction approach involves the use of scaling laws that relate crater dimensions for some reference energy yield to crater dimensions for any energy yield. The reference nuclear yield normally used is 1 kiloton (kt), which is approximately equivalent to the energy released by the explosion of 1 kiloton (2,000,000 lb) of TNT. The results of cratering experiments to date have led to the development of an empirical scaling law based on a scaling exponent of 1/3.4 (4). Figure 1 shows cratering curves (based on the empirical 1/3.4 scaling relationship) that relate apparent crater radius and apparent crater depth to the depth of burst for detonations in hard rock and desert alluvium (1). Similar crater prediction curves (chemical explosives only) have been developed for clay shale (Fig. 2).

EXPERIMENTAL CRATERING PROGRAM

The NCG chemical explosive cratering experiments are designed to calibrate new geologic media for crater dimensions and to serve as forerunners to nuclear experiments in the same or similar media. They are also used to develop techniques of explosively achieving a desired crater geometry. In the case of Project Tugboat and other chemical explosive projects under consideration, they are intended to provide a useful portion of a planned civil works project. One or more of the craters produced in each new medium have been conventionally excavated during post-shot investigations and holes drilled in and around the craters to study the properties of the material immediately surrounding the apparent crater. In those cases deemed appropriate, ground shock and airblast effects have been measured. Radioactive tracer studies have been accomplished in an attempt to compare radioactivity venting from single and multiple-charge events. In short, the chemical explosive experiments continue to provide pertinent data in an expedient and relatively inexpensive manner. The chemical explosive cratering experiments, therefore, do complement the large-yield nuclear cratering experiments.

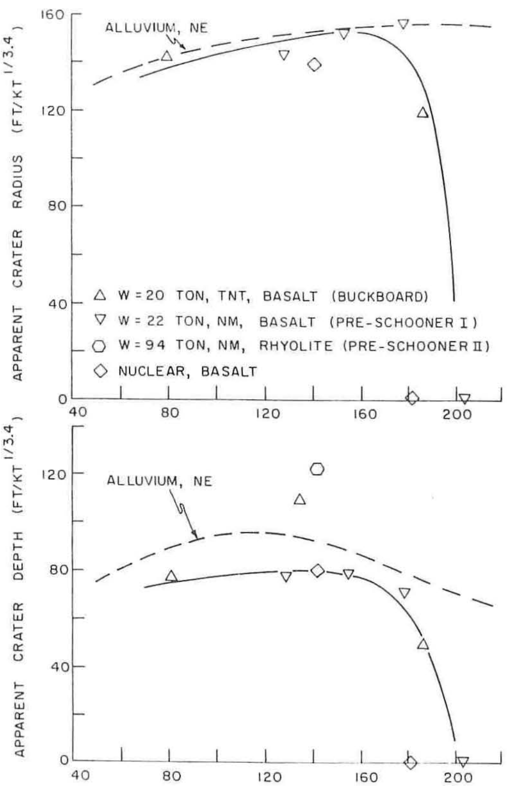

Figure 1. Empirical cratering curves for basalt and desert alluvium.

NCG has executed seven major chemical cratering experiments to date. They are Pre-Buggy I, Pre-Schooner I, Pre-Schooner II, Pre-Gondola I, Pre-Gondola II, Pre-Gondola III, and Phase I of Project Tugboat. All of these experiments except for the last phase of Pre-Gondola III and Tugboat have utilized the liquid explosive nitromethane in spherical containers or cavities. In Phase III of Pre-Gondola III and in Tugboat, an aluminized ammonium nitrate

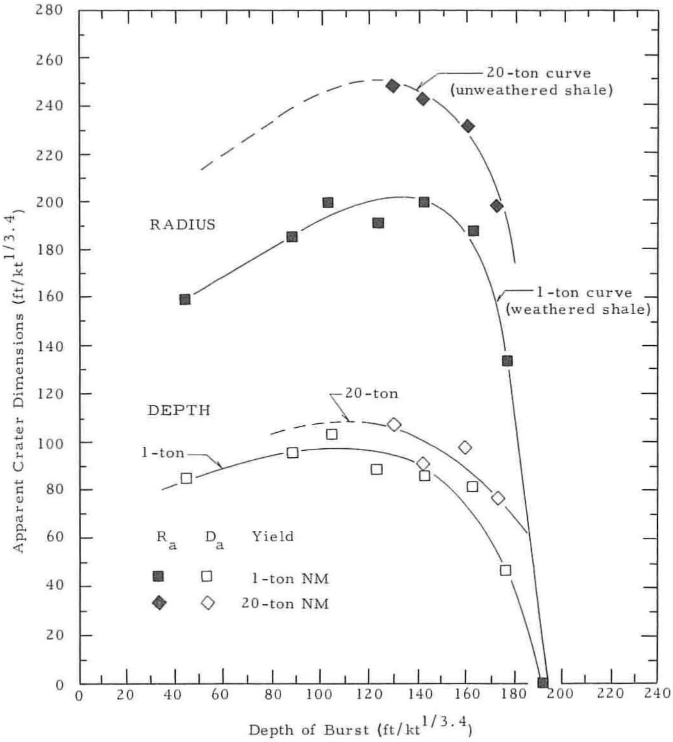

Figure 2. Empirical chemical explosive cratering curves for clay shale.

slurry explosive was used. Yields have ranged from 1,000 pounds per charge to approximately 100 tons per charge.

Project Pre-Buggy I (5) was conducted in alluvium at the Nevada Test Site and included both the detonation of single-charge cratering events and multiple-charge row cratering events. The purpose of this experimental series was to provide data for linear row crater geometry as compared to single-charge crater geometry and to establish design criteria for spacing and depth in row craters. Enhancement in depth and width of the row craters compared to single-charge craters was found. As anticipated, spacing and depth parameters were found to be very interdependent in determining final crater geometry. A spacing of one single-charge crater radius at about optimum depth produced relatively smooth uniform linear craters with no significant cusping. This series was followed by Pre-Buggy II, which further defined row-charge cratering parameters and provided design criteria that assisted in the design of Project Dugout, a chemical row-charge experiment in basalt, and the 5 kt nuclear row-charge cratering experiment, Project Buggy (February 1968).

Project Pre-Schooner I (6) was a series of four 20-ton single-charge chemical cratering events detonated at varying depths of burst in a dry basalt at the Nevada Test Site. These events established an empirical cratering curve for a hard, dry rock. Project Pre-Schooner II (7) was a nominal 100-ton single-charge cratering event in a rhyolite medium in the Bruneau Plateau area of southwestern Idaho. The data from the Pre-Schooner events have been used in the design of the nuclear cratering events Sulky, Cabriolet, Buggy, and Schooner.

The Pre-Gondola experiments were designed to provide crater geometry data in a weak, saturated clay shale. The site selected for these experiments is located adjacent to the Fort Peck Reservoir, Fort Peck, Montana. A number of experiments have

been conducted at the site during the past three years. These have included small-scale experiments in single, row, and array emplacement configurations (8, 9). Yields have ranged from 64 to 2,000 pounds per charge. All of these experiments were peripheral to the main row charge experiment at the site, which is shown in Figure 3 prior to the last row charge detonation. This photograph shows the 20-ton Pre-Gondola I single charge craters (10), the Pre-Gondola II row at the left center, (11) and the Pre-Gondola III Phase II connecting row at the right center (12). Project Pre-Gondola I, four 20-ton cratering detonations, provided data on the variation of crater dimensions in clay shale with respect to depth of burst. The Charlie crater was partially filled in by the Pre-Gondola II five-charge row and is located at the extreme left of the long row crater. Pre-Gondola II consisted of two 40-ton charges and three 20-ton charges spaced at approximately 80 feet and buried at 150 $\text{ft/kt}^{1/3.4}$ (48.8 to 59.9 ft). All five charges were detonated simultaneously to give the linear channel. A wide trench was cut through the side lip and holes drilled into the rupture zone. Pre-Gondola III Phase II provided the longest portion of the crater and consisted of seven charges, 30 tons each, all buried at the same elevation but with variable spacing between charges. Four of the charges were spaced at an average single charge crater radius. The remaining three charges were spaced at 0.6 times the single charge crater radius. That is, the spacing between charges varied and was dependent on the average of the single charge crater radii that would result from the two adjacent charges if detonated separately as single charges. This charge configuration gave a very smooth, large crater that connected to the Pre-Gondola II crater. The Pre-Gondola I single charge detonations were executed during the fall of 1966, the Pre-Gondola II row during June 1967 and the Pre-Gondola III Phase II row during October 1968.

The last major experiment in the Pre-Gondola series was executed on October 6, 1969. This was Pre-Gondola III Phase III, Reservoir Connection Experiment. In this experiment, five charges of varying yield and depth were so placed and simultaneously detonated to provide a connecting channel between the long crater shown in Figure 3 and the Fort Peck Reservoir. A centerline section drawing showing individual charge yields and locations is shown in Figure 4. Figure 5 shows the preshot view of the crater. Just after the detonation, Figure 6, water started to fill the crater. The water filling action, Figure 7, took about 9 minutes. The final view, Figure 8, shows what the crater looked like when filled to reservoir level. The crater width at water level

Figure 3. Pre-Gondola 20-ton single-charge craters and connecting row crater prior to execution of the reservoir connection experiment.

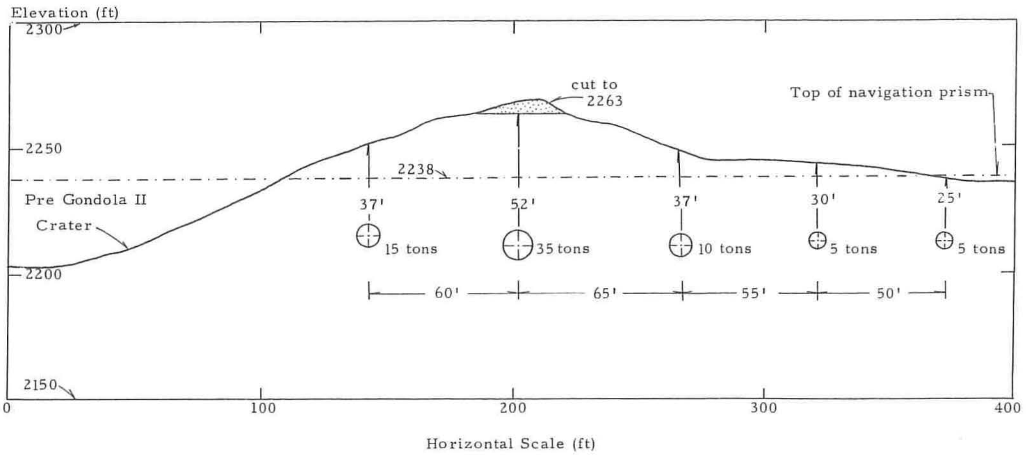

Figure 4. Centerline section drawing of the reservoir connection experiment, Pre-Gondola III, Phase III, showing charge depths, spacings, and yields.

Figure 5. Preshot view of the Pre-Gondola III, Phase III reservoir connection experiment.

varies from a minimum of 100 ft to a maximum of 200 ft. The depth of water in the crater varies from a minimum of 13 ft to a maximum of 39 ft, except at the entrance where the depth is approximately 7 ft. The length of the water-filled portion of the crater is approximately 1,370 ft. Although this work was totally experimental, it was very successful and graphically illustrates two proposed applications of large-scale explosive excavation, an inland harbor and a canal.

Interest in the explosive excavation of harbors has generated the most recent chemical explosive cratering project being conducted by NCG, known as Project Tugboat. This explosive excavation experiment is designed to investigate the general concept of producing a harbor basin in shallow water in a near-shore environment. The site for the experiment was picked to coincide with the site of a planned small boat harbor so that some benefit would be obtained from the expenditure of the research and development funds. This site is in Kawaihae Bay on the west side of the Island of Hawaii (Fig. 9). The project is planned for execution in three phases. Phase I, executed November 4-7, 1969, was a cratering and safety calibration series of detonations. Phases II and III are planned as row or array detonations of nominal 10-ton charges designed to excavate a berthing basin and entrance channel.

Experience in cratering in a completely saturated medium overlain by water is almost nonexistent. Because of this, five detonations were included in the Phase I program, four each 1-ton and one 10-ton. The 1-ton charges were placed at depths ranging from 16 to 24 ft below mean low water level. This program was intended to provide crater dimension and safety data as a function of both depth of burst and yield.

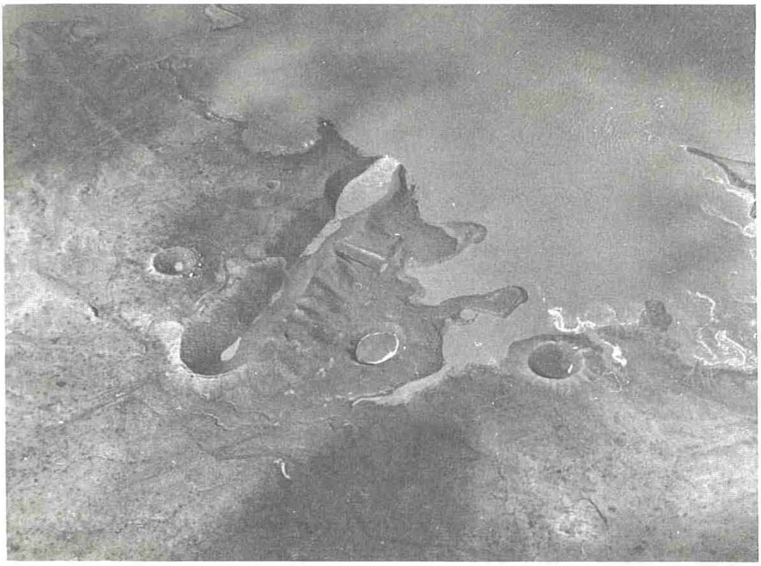

Figure 6. Reservoir connection experiment crater immediately following the detonation, showing water starting to fill the crater.

Figure 7. Reservoir connection experiment crater filling with water.

Figure 8. Reservoir connection experiment crater after water-filling action was complete.

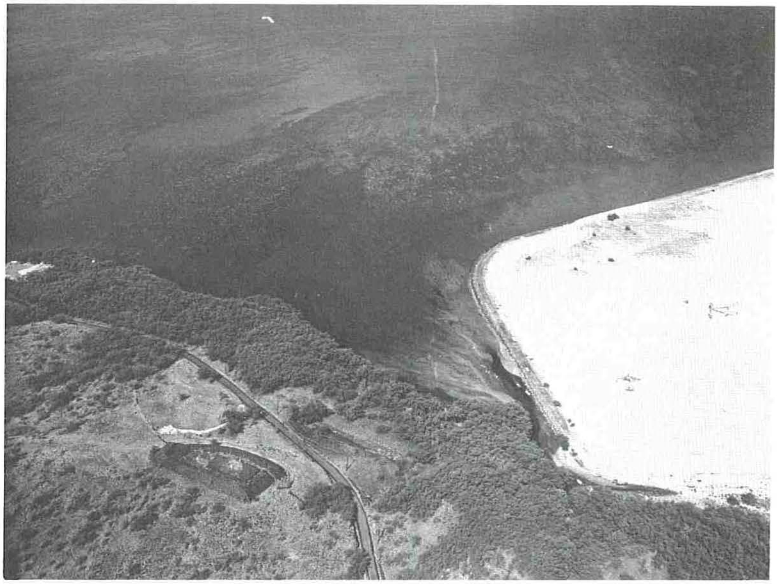

Figure 9. Site for the planned Kawaihae small boat harbor. The harbor is to be located in the coral reef area in the upper right quadrant of the picture.

The site medium is a coral limestone extending to 70 ft or more in depth and overlain by 6 to 10 ft of water. The original concept for explosively excavating a harbor in this material assumed that the crater formation process would be similar to that experienced in previous dry-land experiments and that a crater lip would form that could be used as the core for a breakwater. After laboratory testing data were obtained for

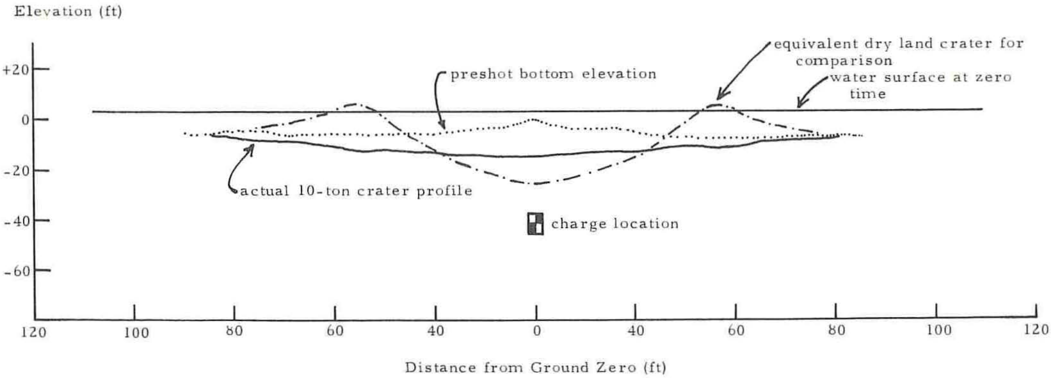

Figure 10. Profile of Project Tugboat, Phase I, 10-ton crater shown in comparison to an equivalent-yield dry-land crater profile.

the coral, it was evident that the concept might be somewhat in error. The porosity of the material ranged from 37 to 64 percent, and the compressive strength was variable and ranged from 760 to 1,738 psi. The data strongly indicated that the material would be compacted in the cratering process and very little ejecta would be available to form a lip that would extend above water. This indeed was the case for both the 1-ton and 10-ton craters. A profile of the 10-ton crater is shown in comparison to a dry land crater in Figure 10. As can be seen, there were no lips. The total apparent crater volume seems to result from crushing and compaction of the coral. The crater shape is more desirable for creating a harbor than that originally contemplated based on dry land experience in that it is very broad and of shallow depth. On the basis of these calibration results, it is currently estimated that the harbor basin and entrance channel can be accomplished with about half the amount of explosives called for in the original design. At this writing, data analysis is still proceeding.

ENGINEERING PROPERTIES OF NUCLEAR CRATERS

NCG technical participation in AEC nuclear cratering experiments has included crater measurements, a joint long-range fallout monitoring and interpretation program with the U. S. Public Health Service and the Lawrence Radiation Laboratory, and post-detonation excavation and drilling of the crater fallback, ejecta, and rupture zones for engineering properties investigations.

Engineering properties investigations at the nuclear cratering experiment sites have been performed by NCG in a manner similar to those developed in the postshot excavation and drilling of the chemical explosive craters. These studies are intended to provide information that will permit an evaluation of the usefulness of the crater as an engineering structure or of crushed and broken rock as usable quarry rock.

A nuclear detonation in soil or rock produces significant changes in the media surrounding the visible crater. To assess the engineering usefulness of the crater, one must be able to predict the extent and physical characteristics of the zones of disturbance created by the detonation. The nature of these zones affects such engineering considerations as stability of crater slopes, foundation conditions in the vicinity of the excavation, and seepage and drainage in the media altered by the detonation. A significant portion of the nuclear excavation research program, therefore, is devoted to determination of how nuclear cratering detonations affect the immediate geologic environment and the impact of cratering formation phenomenology on the stability of crater slopes.

The results of nuclear crater properties investigations to date indicate that the disturbed zones surrounding the crater may be categorized as follows (Fig. 11):

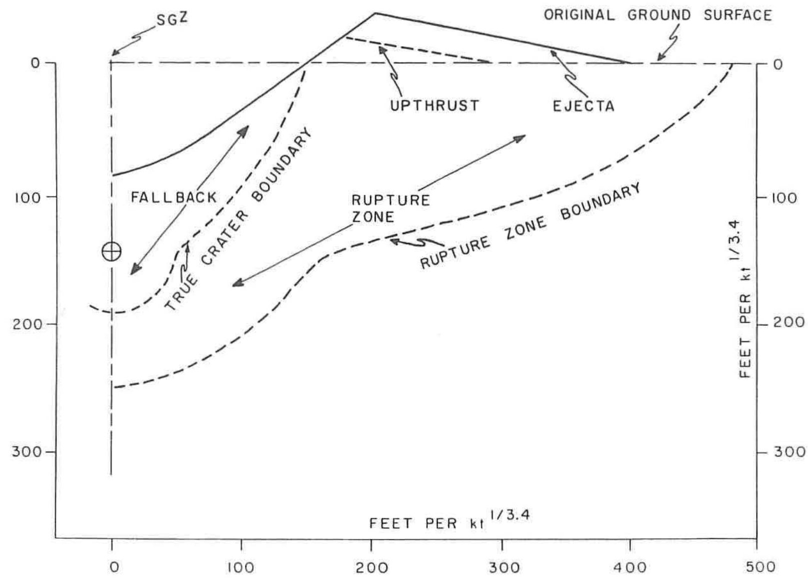

Figure 11. Cross section of typical crater in hard rock showing zones of disturbance.

The apparent crater is defined as that portion of the visible crater which is below the preshot ground level.

The true crater is defined as the boundary (below preshot level) between the loose, broken, disarranged fallback material and the underlying material that has been crushed and fractured but has not experienced significant vertical displacement or disarrangement.

The fallback consists of materials that have experienced significant disarrangement and displacement and have come to rest within the true crater.

The rupture zone is that zone extending outward from the true crater in which stresses created by the detonation have caused fracture and crushing of the material. In this zone, displacements and changes in density are evident but the material remains basically coherent in contrast to the disarranged fallback materials.

The elastic zone is that zone extending beyond the rupture zone in which no fissures, cracks, or permanent displacement of material are evident. Strong earth motions are propagated through the elastic zone to great distances.

The ejecta consists of material thrown out above and/or beyond the true crater.

In order to analyze effectively the potential engineering behavior of an excavation produced by nuclear explosives, one must be able to predict with reasonable accuracy the following geometric and physical characteristics of the various crater zones:

1. Geometry of the apparent and true craters;
2. Geometry, effective porosity, bulk density, permeability, and in situ strength characteristics of the rupture zone, fallback, and ejecta;
3. Particle size of the fallback and ejecta; and
4. Degree and orientation of blast fracturing in the rupture zone.

Techniques for predicting apparent crater geometry have been described in preceding sections of this paper. The general shape of the rupture zone for craters produced by nuclear explosives buried in the optimum depth of burst region is shown in Figure 11 (13).

The comparison of preshot in situ block or grain size with the particle size of the ejecta and fallback for the craters investigated to date has shown a fairly close

correlation. In a moderately to highly fractured rock medium, the preshot in situ block size has a significant influence on the fallback/ejecta sizes. There are also indications that increasing depths of burst may tend to increase the percentage of coarse particles. As a rock medium becomes more massive (i.e., spacing between joints and fractures greater than 3 to 5 ft), the degree of control of in situ block sizes on fallback and ejecta sizes becomes less pronounced.

Information from investigations completed to date indicates that the bulking factor of the fallback and ejecta material (ratio of preshot bulk density) will be within the range of 1.1 to 1.6 for hard rock media.

The limit of blast fracturing and the outer boundary of the rupture zone are coincident. Comparison, to date, between the limit of bulking (zone of increased effective porosity) and the limit of blast fracturing indicates that their envelopes are also nearly coincident. Both the intensity of blast fracturing and bulking decrease with distance from the true crater boundary. The observed concentrations of blast fracturing and relatively high effective porosity appear to extend along boundaries between different rock types.

Postshot field investigations of the Cabriolet nuclear crater (2.3 kt at 171 ft in rhyolite/trachyte) have recently been completed (14). A trench was excavated through the lip of the crater and the material screened and weighed. The measured bulk density of the ejecta was 124 lb per cu ft, giving a bulking factor of 1.10. The true crater radius was estimated at 210 ft. This is about 18 ft larger than that predicted by Figure 11. The maximum uplift of the rupture zone observed in the excavated trench was 14 ft.

Because of the high cost of obtaining this kind of information for the large nuclear craters planned for execution in the research program, NCG has initiated an effort to develop the capability to obtain the required information by large-diameter core drilling of the fallback, ejecta, and rupture zone. The initial effort will be to develop a disintegrating grout that can be used during coring operations but can be easily separated later from the cored material to permit determination of a size gradation curve and bulk density measurements.

Techniques have not been developed that would enable one to predict accurately the permeability and in situ strength characteristics of the fallback, ejecta, and rupture zone. The development of the required additional prediction techniques and the refinement of the existing techniques as discussed here are being accomplished under the research program.

NCG STUDIES OF ENGINEERING FEASIBILITY

Studies of the feasibility of using nuclear explosives for specific civil works projects serve three purposes. First, they provide a feedback to the research program of specific problem areas encountered in real applications. Second, because they are being accomplished by engineers in the Corps Districts throughout the United States, they are serving to train a large pool of talent in this new technology. Third, they serve to develop experiments that may be used in conjunction with the construction of actual civil works projects to demonstrate the viability of nuclear excavation.

The primary purpose for the studies has changed from the initial one, which was to identify specific problems that could be solved in the research program, to identifying specific projects that could be accomplished using nuclear explosives. Twelve studies of specific civil works project applications have been completed or are near completion. They have included studies of spillway nuclear excavation, canal nuclear excavation, nuclear quarrying, creation of dams with nuclear explosives, and nuclear harbor excavation.

During the course of these studies, many problem areas were identified. One problem identified early concerned nuclear explosive emplacement construction. Studies were initiated to determine the best techniques and the costs of drilling large-diameter (>30 in.) emplacement holes in varying geologic media (15). Also initiated were studies of methods and costs of constructing emplacement holes in disturbed materials in and near existing nuclear craters for extending a nuclear excavation (connecting charge)

or modifying an existing crater (triple row technique, second pass emplacement). A study was also initiated on conventional excavation techniques for use in conjunction with nuclear excavation projects (16).

Early in the feasibility study effort it became apparent that an analytical technique for predicting gamma radiation exposure rates in the nuclear crater and lip area as a function of time was needed. In most of the projects studied there was a need to reenter the crater area as early as practical to carry out conventional construction activities. A prediction technique has been developed (17) that assumes mixing of the radionuclides produced with a portion of the volume of material making up the true crater volume. A significant conclusion of this work is that, for cratering detonations at optimum depth of burst, reentry times decrease as explosive yield increases. This conclusion is shown in Figure 12. The assumptions made to arrive at this prediction are that the fission portion of the nuclear explosive yield is 3 kt and does not change with yield and the gamma-emitting induced radionuclides that contribute to the exposure rate in the crater are the same as those given in AEC Classification Bulletin WNP-11 for radionuclides present in the radioactive cloud and fallout from a Plowshare cratering detonation. Also, induced radionuclides in cloud and fallout are in the same ratio to the total produced as the fission products given in WNP-11 are to 3 kt assumed for the explosive. The dose rate assumed safe for reentry is 2.5 mR/hr. With expected improvements in explosive design, it is predicted that entry can be made to a megaton-yield crater with proper rad-safe control approximately 3 weeks following the detonation.

NUCLEAR CONSTRUCTION APPLICATIONS

The potential use of nuclear methods of construction (18) covers a wide range of projects. It is reasonable to anticipate that nuclear explosives could be used advantageously in the construction of such water resource projects as navigable waterways, dams, harbors, storage reservoirs, or spillways. In addition, nuclear-excavated cuts could be incorporated in highway and railroad construction to provide rights-of-way through mountainous or precipitous terrain. A rather basic application of nuclear construction techniques would involve the detonation of a nuclear explosive at a relatively deep depth of burst to produce aggregate for use in the construction of dams, breakwaters, and other rockfill structures.

The following paragraphs describe those nuclear applications which are considered to have the greatest potential for use in the construction of large-scale civil works projects.

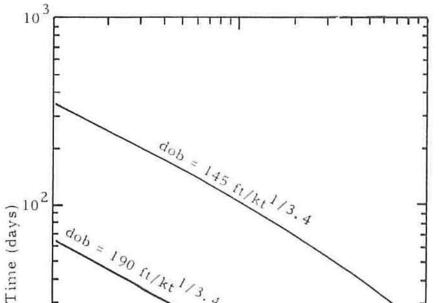

Figure 12. Predicted reentry time into the gamma radiation field in the nuclear crater ejecta and fallback region as a function of total nuclear explosive yield for two depths of burst.

Nuclear Quarrying

The subsurface detonation of a nuclear explosive has potential for producing a large volume of broken rock at a low unit cost. The basic concept in using nuclear explosives for quarrying purposes is to detonate the device at such a depth that the quantity of broken rock is maximized and the distance to which the rock is ejected is minimized. In order to facilitate removal of the rock after the detonation and to facilitate subsequent operations of the quarry, a sloping terrain configuration appears to be the most advantageous topographic environment for nuclear quarrying projects.

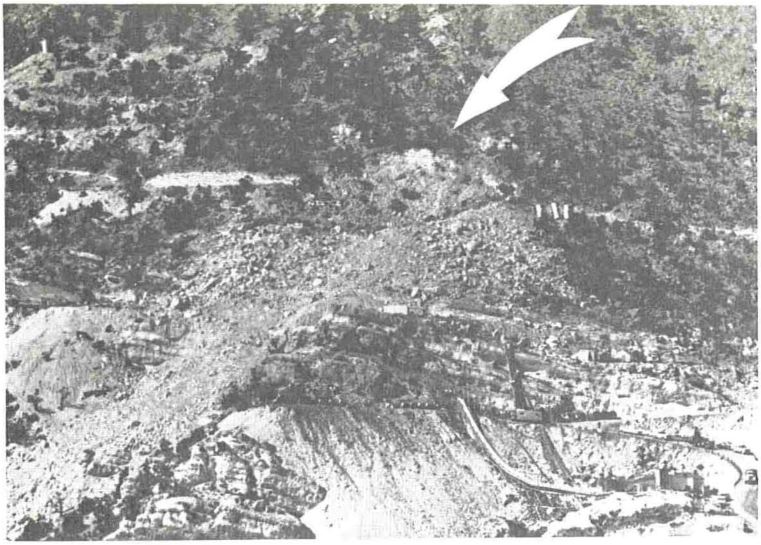

Figure 13. Neptune postshot configuration.

Several of the nuclear cratering experiments that have been executed to date have provided information of significant value in developing nuclear quarrying technology. The Neptune Event was a 115-ton nuclear detonation at a depth of 85 ft under a 30-deg slope in 1958. The resulting postshot configuration is shown in Figure 13. About 34,000 cubic yards of material were ejected downhill as a result of this detonation.

In December 1964, the Sulky Event (85 tons nuclear at a depth of 90 ft under level terrain) was detonated in basalt at the Nevada Test Site. The resulting configuration (Fig. 14) was a mound of rock that projected above the preshot ground surface rather than the classical crater.

As currently envisioned, nuclear quarrying projects would involve detonation of a nuclear device under sloping terrain at a depth of burst similar to that used in the Sulky detonation. The broken rock resulting from such a detonation could be removed with reasonable ease and used as rockfill for dams, breakwaters, or other construction projects requiring large quantities of aggregate. A concept of a nuclear quarry in operation is shown in Figure 15.

Figure 14. Sulky Event.

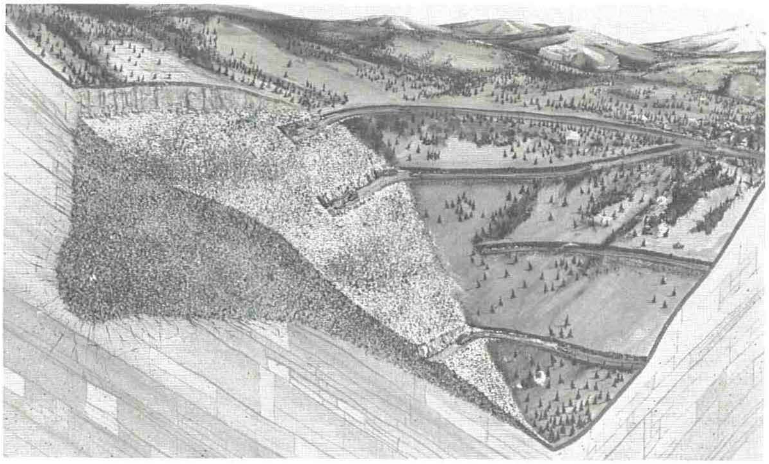

Figure 15. Nuclear quarry concept.

Nuclear Ejecta Dam

A potential nuclear construction application that appears to be quite feasible involves the detonation of a nuclear explosive in the wall of a canyon to eject material across the canyon and thereby create a water storage embankment. In addition to the material actually ejected into the canyon, it is reasonable to assume that some material would collapse from the region immediately above the true crater boundary and add to the total volume of embankment material.

The technique of using explosives to create dams across canyons has been successfully demonstrated by the Soviet Union. The detonation of 2,000 tons of chemical explosive in a narrow, steep-walled canyon on the Vakhsh River in Tadzhikistan resulted in a 2.6 million cu yd rock-fill dam.

In addition to the nuclear detonation itself, consideration must be given to the practical engineering aspects of the dam construction such as an impermeable embankment seal, settlement of the ejecta material, and seepage through the embankment. It is

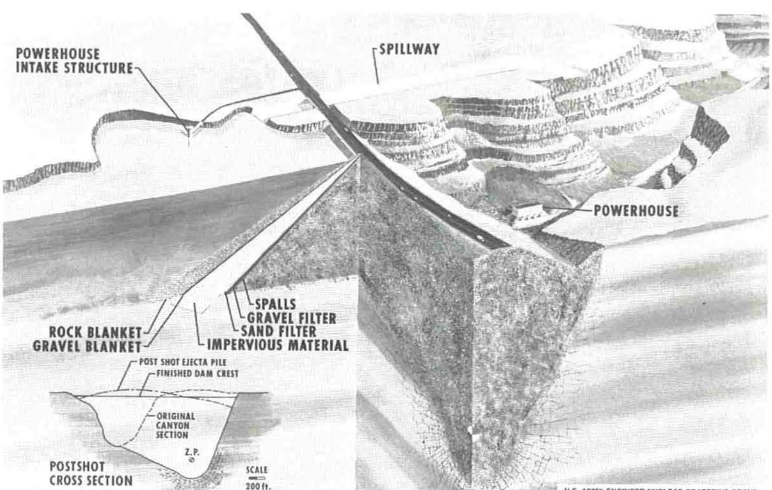

Figure 16. Nuclear ejecta dam concept.

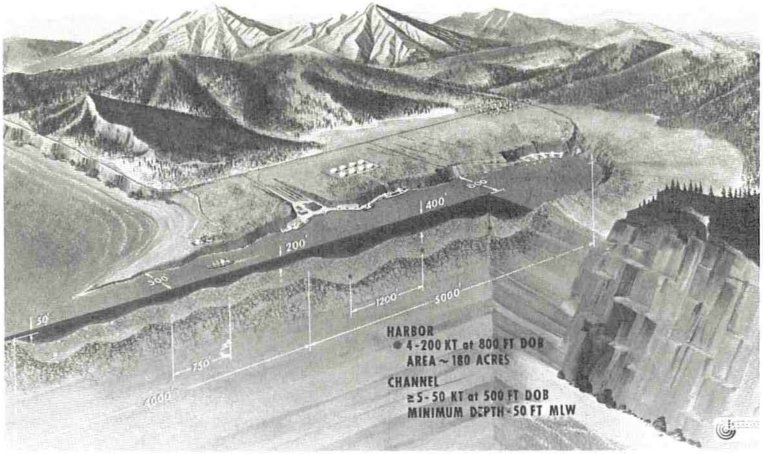

Figure 17. Harbor excavation concept.

planned that these engineering problems would be investigated in a chemical explosive experiment on a smaller scale that would precede a nuclear experiment. The concept of a nuclear ejecta dam is shown in Figure 16.

Nuclear Harbor

The concept of using nuclear explosives to produce protected water areas of sufficient depth to facilitate entry, unloading, and exit of deep-draft vessels has been considered for several years. The crater formation process, in addition to creating an excavation of the required depth, results in the formation of a crater lip that may well function as a breakwater to protect the harbor area from wave action. The nuclear construction aspect of the harbor may involve the detonation of a single explosive to produce the harbor area itself as well as the detonation of a row of explosives to excavate the entrance channel. A concept of this application is shown in Figure 17.

The conventional engineering and construction aspects of harbor design that must be considered in conjunction with the nuclear aspects are most important. Loading and unloading facilities must be constructed as well as areas for cargo clearance and vessel anchorage.

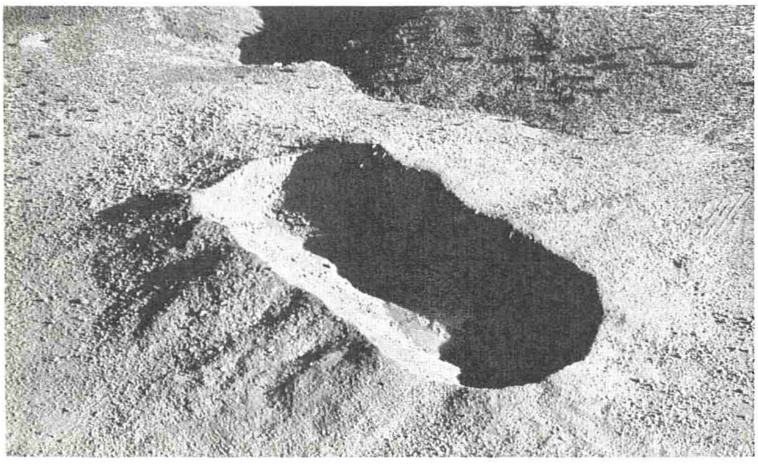

Figure 18. Buggy crater.

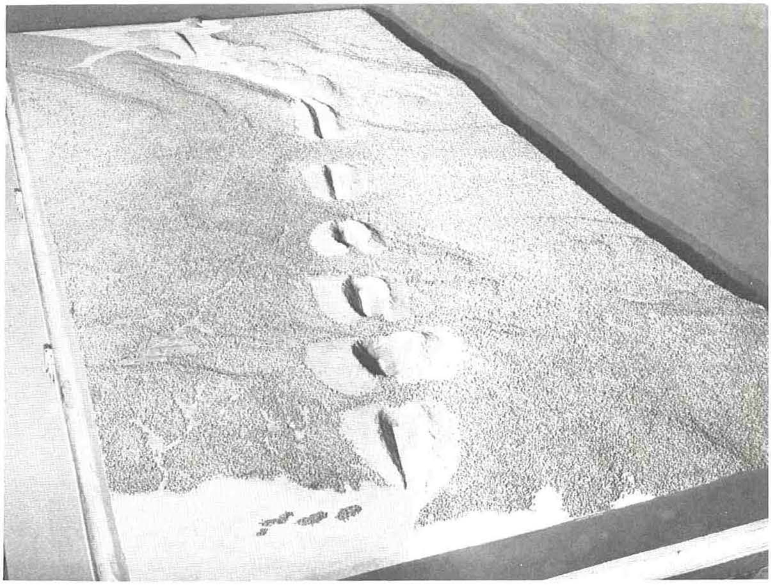

Figure 19. Model of transisthmian sea-level canal nuclear excavation showing results of first-pass detonations.

Nuclear Excavated Cuts

A series of nuclear explosives may be detonated simultaneously in a row to produce a linear crater. The linear crater may be used, in turn, as a navigable water way or canal or as a right-of-way for a highway or railroad through mountainous terrain.

The Buggy Event, March 12, 1968, consisted of the simultaneous detonation of five 1.1-kt nuclear explosives and resulted in a linear crater approximately 900 ft long, 250 ft wide, and 60 ft deep. The postshot configuration of the Buggy crater is shown in Figure 18.

Figure 20. Artist's sketch of a nuclear-excavated canal through the Darien region of eastern Panama.

The most widely known potential application of nuclear explosives in the excavation of a navigable waterway is the proposed construction of a sea-level canal through the Central American isthmus. The excavation of such a canal by nuclear methods would involve the detonation of a series of linear craters rather than a single linear crater. The total nuclear yield for the excavation across the entire isthmus would be excessive for safety reasons if detonated all at one time. Figure 19 shows an alignment across the Central American isthmus subsequent to the detonation of the first series of linear craters. The second pass of detonations would produce linear craters that would connect to those produced during the first pass and thereby result in a continuous sea-level waterway from the Atlantic to the Pacific Oceans. A concept of a portion of the completed nuclear-excavated sea-level canal along an alignment in the Darien region of Panama is shown in Figure 20.

The Nuclear Cratering Group is currently developing nuclear excavation designs for proposed sea-level canal alignments in conjunction with the current Atlantic-Pacific Interoceanic Canal Studies.

Other potential uses of nuclear-excavated cuts include the construction of spillways at a site remote from a dam, river diversion channel, or reservoir outlet canals.

SUMMARY

The NCG program activities include (a) cratering calibration of differing geologic media and testing of techniques designed to provide a desired crater geometry with chemical explosive detonations; (b) joint planning of, and technical participation in, AEC nuclear excavation experiments; (c) development of pertinent data on the engineering properties of nuclear craters; (d) development of civil works chemical and nuclear explosive construction technology; (e) accomplishment of engineering studies of nuclear construction feasibility; and (f) execution of joint CE/AEC civil works nuclear construction experiments.

The chemical explosive tests conducted thus far have provided empirical data for the design of the nuclear experiments Sulky, Cabriolet, and Buggy. The Pre-Gondola experiments have provided cratering experience in a wet clay shale and in the row charge and connecting row charge techniques.

Engineering properties investigations of nuclear craters are providing data that will be used as a basis for assessing the suitability of a nuclear crater for the engineering applications presently contemplated.

Four conceptual nuclear construction applications have been identified as having a significant potential for accomplishment: (a) nuclear quarrying to produce rockfill or aggregate; (b) nuclear ejecta dam construction; (c) nuclear harbor construction; and (d) nuclear canal excavation. The nuclear quarry has been identified as the most direct application of present technology.

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Tunneling Machines of Today and Tomorrow

T. N. WILLIAMSON, Jacobs Associates, San Francisco

Current tunneling machines are competitive in making circular bores in rock as strong as 25,000 psi compressive strength and in diameters to 20 ft. These machines produce tunnels 50 to 100 percent faster than conventional ones, and show tremendous savings in permanent tunnel linings. Machines of the future will be able to cut other than circular bores and be competitive in many formations as strong as 35,000 psi strength and in sizes equivalent to 35 ft in diameter. Principal development is required in rock disintegration, material handling, and temporary roof supports.

• THE INCREASING public awareness that population concentrations demand more underground facilities has spurred a tremendous interest in tunnel boring by machine methods. The need for rapid transit systems, the desire to reduce the number of unsightly elevated freeways, the need for more parking facilities, the requirements of the civil defense, and the high cost of urban surface real estate all point to a greater demand for improvements in underground excavation technology.

The shallow or top few hundred feet of earth crust will contain many of the public works tunnels. This crust is not uniform so several methods of boring will be required, sometimes within the same tunnel. This discussion will deal principally with so-called "hard-rock" tunnels, a term loosely applied to any rock much stronger than well-prepared plaster of Paris. It will ignore that very important field of soft-ground tunnels where shield driving (Fig. 1) is a well-advanced art. More than 60 of these shields have been built in the United States.

Rock tunnels are driven by drill and blast and by tunnel-boring machines (TBM). The first successful TBM's in 1954 were for 26-ft diameter soft-ground tunnels at Oahe Dam in South Dakota. They remain the largest rock machines used to date in the United States. One larger, 36-ft, James S. Robbins' TBM was used on the Mangla Dam in Pakistan by the Guy F. Atkinson Company.

The South Dakota machines were developed from the technology borrowed from the developments of continuous coal-boring machines. The coal industry now has more than a thousand continuous miners in use, most of which have been developed since World War II. The first South Dakota machine was built by Robbins for the contractor, Mittry, and is frequently called the Mittry machine. This TBM used drag cutters to cut kerfs and discs to split the ridges between the kerfs after they had built up. This method is still used by some machines in coal boring. Even before the Mittry machine was built, the U. S. Army Corps of Engineers had supported the development of a coring or a gage kerf-cutting device for the Oahe shales. The gage kerf was cut with a coal-mining machine, which resembles a large chain saw.

While the Mittry TBM was being developed, independent developments for harder rock were being carried out in shaft sinking in West Virginia, Germany, and Holland and on a tunnel borer in England. The Dutch and Germans were boring 26-ft diameter coal-mine shafts (Fig. 2), and the Germans were developing the first raise drills to connect overlying coal-mine entries below ground. One German developer (Salzgitter) tried to build a shaft drill that cored from the bottom up. The German Bade made a

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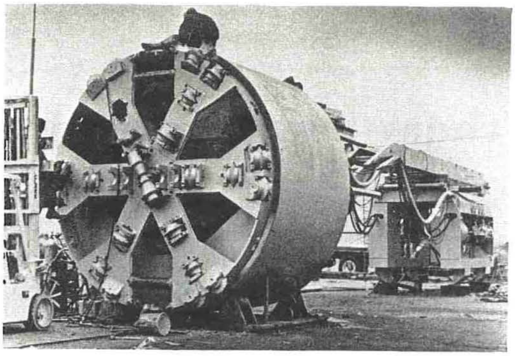

Figure 1. Soft-ground tunnel driving shield with 13 ft 4 in. diameter disc cutters.

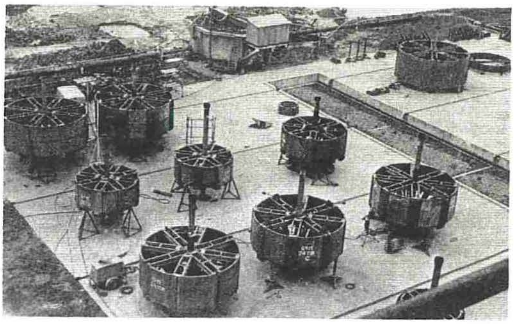

Figure 2. Bit bodies for Dutch rotary reaming 26-ft diameter mine shafts.

drill for shaft sinking using a unique principle of rolling cutters turning in a planetary action about a central rotating shaft. This principle is being modified and tried in a new approach to tunnel boring machines in England, Germany, and Switzerland, as will be explained.

The Zenis in West Virginia, with assistance from Hughes Tool Company, developed 2 machines to drill 6-ft diameter mine shafts during the 1950's. These machines used rolling cutters, and the first of the two was a core drill.

Hughes built a horizontal test TBM in the late 1950's and with it proved that rock harder than 35,000 psi compressive strength could be drilled in rather large diameters, but on a laboratory scale. At about this time or in the early 1960's, Robbins put his discs closer together, eliminated his drag cutter, increased his thrust, streamlined his machine design, and successfully drilled some rock of about 12,000 psi strength. This was the first application of discs as the primary cutter to large diameter rock drilling. A little later, K. C. Cox of Dravo, with assistance from Hughes, showed that discs on a pointed or conical head could be made to break more of the rock in tension and thus reduce horsepower and thrust requirements.

Several machines were then built from 80 in. to 20 ft in diameter (Fig. 3). TBM's were applied progressively to harder rock and are now being used in 22,000-psi limestone in Chicago. Some of the layers of rock being bored at White Pine Copper exceed 30,000-psi compressive strength, but the TBM application in rock this strong must await a cutter cost reduction below that estimated today before it can be considered a complete commercial success in public works tunnels.

It should be pointed out that, while compressive strength is the best rock characteristic to use in estimating its borability, it is not conclusive. Compressive strength is difficult to measure precisely, partially because natural flaws exist even in apparently homogeneous rock. Rocks of equal compressive strength will bore differently depending on brittleness and other factors. Limestone of 20,000 psi generally will drill easier than a tougher (less brittle) schist of the same strength. Rock constituents also will affect cutter life and cost. Rock containing a large percentage of quartz will wear cutters faster than that having predominantly a less abrasive mineral such as calcite.

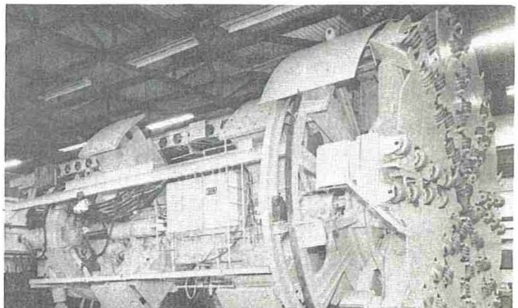

Figure 3. Hard rock boring machine 20 ft in diameter that bored 7,800 ft in BART Tunnel at 60 ft per day.

There are 4 manufacturers of rock TBM's in the United States and 3 in Europe. Most of them apply thrust of at least 50,000 lb per foot of diameter from wall anchors. The Lawrence Manufacturing Company machine gets a major portion of its thrust by pulling from an anchor set in a 24-in. predrilled pilot hole. Most of them use an essentially flat face on the cutter head, although the Robbins has a slight ovoidal or saucer shape. The cutter head is rotated at approximately 80 rpm divided by the diameter in feet. The rotary horsepower is approximately 50 times the diameter in feet, and most of them have 100 to 150 auxiliary horsepower. All use electric power, and all transfer some of this to hydraulic power for thrust; some use hydraulic motors for the rotary drive.

One British machine, the McAlpine, uses a planetary action on a movable cutter head and can thus cut a heading of any desired shape. This cutter head, like that of the old German Bade shaft-boring machine, has some drag action to the cutters and thus can be considered only with caution for strong, tough, abrasive rock because of rapid cutter wear and slowness of penetration. The Swiss Habegger and Wholmeyer machines have a similar action but have not been used in this country or elsewhere extensively because of this and also because of their mechanical complication. Krupp in Germany did some experimental work with this principle.

Some who analyze machines with planetary action point out that cutters are used in a chipping as well as a drag action. It is also suggested that some of them cut radially from a predrilled hole, that they require less thrust, or that they cut rock in tension rather than compression. On close examination none of the arguments can be substantiated. First of all, the pilot hole must be drilled, and this is time consuming. Then the rock under radial attack must be chipped under compression, and most of such rock being attacked responds similarly to attack from north or south as it would from east or west. The main difference is that the forces resisting the thrust on the rock must be taken by the machine in the radial attack, whereas in the frontal attack they are taken by the interface of the machine to the tunnel wall. This is a doubtful advantage and, if one exists, it is probably outweighed by the frontal attack covering more face area at one time with much less complication of machinery.

It may be overlooked that all rolling cutters that have teeth provide a chipping action. Those that rotate in a more usual, nonplanetary motion do not have as much self-destructive drag action as planetary cutters in hard rock. Some drag action is desirable only in very soft formations. The very good advantage of the Bade-McAlpine planetary approach is that it will cut a horseshoe shape so desirable for transportation tunnels.

Nearly all the TBM's that use rolling cutters offer one or more versions of the disc cutter. Some have single discs on a spindle. Some have 3 or 4 discs on each spindle, and the spindle diameters vary from 9 to 15 in. Some have steel edges, and some have sintered tungsten carbide inserts as teeth or wearing surfaces. Some have replaceable cutter shells so that either the bearing or the cutting surface can be replaced. The tooth type of cutters, which were used on the original Zeni rock-boring machines, are seldom used in rock tunnel boring today, but some form of tooth cutter may regain usage as machines move into stronger rock applications.

As previously mentioned, the Germans developed the raise drills early in the 1950's. They drilled holes of about 36 in. in diameter in sedimentary rock. Raise drills are drills normally used in deep mines to drive vertical or sloping shafts between vertically separated tunnels. They are mentioned in this discussion on tunneling because their development may influence hard formation tunneling. The raise drilling rig drills a small hole of 8 to 12 in. to connect the tunnels. A 60-in. bit is put on at the other end, and the hole is backreamed letting the cuttings fall into the lower tunnel. Hughes used the idea in the late 1950's and "beefed up" the European design to cut 60-in. holes in very hard rock at Cleveland Cliffs, Michigan, iron ore mines. Robbins and others subsequently built more than 50 of these machines (Fig. 4). These machines have proved that, by providing sufficient thrust and power, the hardest rock can be bored at the reasonably good penetration rates of 2 or 3 ft/hour. The cutter cost today is reported to be $8 to $12/cu yd or higher. This high cost, combined with the rather slow penetration, all but rules out TBM's for hard rock because drilling and blasting in this material is about as fast, perhaps slightly less costly, and more reliable. It may be interesting to note that in sandstone of about 10,000 psi TBM's have advanced at 17 ft/hour and at cutter costs reported to be less than $1/cu yd.

Today's TBM's differ only slightly in their guidance means. Some machines can change direction while they bore and others reset direction at the end of each stroke. In the latter method strokes may be shortened if necessary for a continuous curve. In any event, most TBM's have good guidance control, and one tunnel was bored within $^5/_8$ in. of the prescribed line and grade. The stroke length on different TBM's has ranged from 1.5 to 5 ft.

All TBM manufacturers offer mechanical aids for setting ring beam supports above or around the machine and within about 5 ft of the face. None of these is completely automatic, and most of them are quite awkward and leave considerable room for improvement.

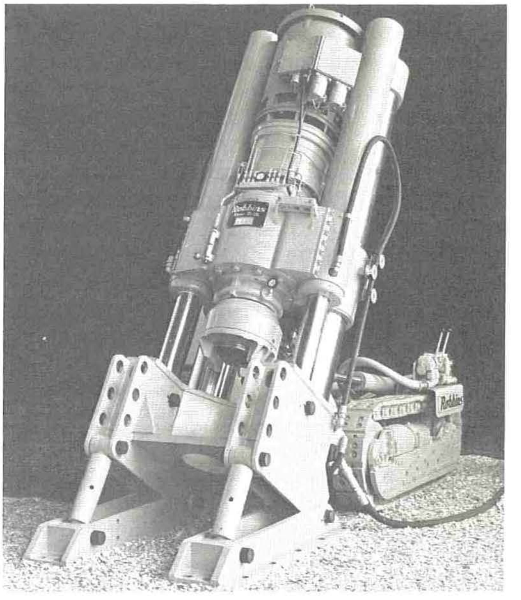

Figure 4. Raise drill for drilling small hole into a rock tunnel and pulling a 60-in. reaming bit up.

Muck is picked up by buckets on the outer edge of the cutter wheel and deposited onto a belt conveyor to be transported to tunnel cars in the rear of the TBM, except in a few machines such as the McAlpine. The muck in any multiple-head machine such as the McAlpine either is plowed onto transverse drag conveyors discharging at the center on a longitudinal belt conveyor that moves it to the rear or is gathered to the central conveyor by revolving arms like those on snowplows or coal-mining machines. Such gatherers will have high maintenance when required to handle sharp-edged, hard, abrasive materials. One of the Lawrence machines used a screw conveyor rather than a belt to move the material out. Where possible, the machine belt conveyor should be at least 30 in. wide to handle peak loads during very fast penetration and to handle large rock particles that fall off the face or roof.

The TBM manufacturers or the contractor must take considerable interest in seeing that the trailing conveyor is adequate and that there is a minimum delay in waiting for cars. More than half of the TBM applications to date have had less than adequate car supply facilities. Except in the very best jobs, delays of more than 40 percent of the available time have been caused by waiting for cars. Trailing conveyors of early tunnel borers were 60 to 150 ft long. Many of those being built today are 300 ft or longer and straddle a double track. Long thin cars are being designed for small tunnels so that a string of empties can be stored under the conveyor alongside that string being loaded to avoid car waiting delays.

No rock TBM manufacturer, or user, has developed a successfully proven method for concrete lining concurrently with the boring. The complication of having forms in the way in a congested tunnel, hauling concrete in without interfering with muck haulage out, and generating added heat so far have been insurmountable problems.

A study of patent files shows that for more than a century, man has dreamed of a tunnel-boring machine. Machines were used in the last century in England and the United States, but none prior to 1953 got much beyond the prototype stage. It has taken nearly 20 years for the current concept of a rock TBM to develop to the present state. There have been no significant innovations in the TBM's in the past decade, other than laser guidance. Backup facilities, such as conveyors and car changers, have been improved as has TBM reliability; but cutters, rpm, and thrust types and techniques generally are the same as were drawn up in 1960 and were in the concept stage in 1955. This is revealed in patents and in some of what appeared to be "visionary" technical papers of that period.

This indicates that with our existing approach it takes at least 15 years to get an idea from the drawing board to complete acceptance in the field. Much of the first 10 years is used convincing a broad segment of the users that the idea is feasible. They are the ones who have to gamble the money and make the ideas work. There are still some who are not convinced that the TBM has "arrived," even though they have lost profitable jobs to those who were convinced that the rock TBM would work. There are some of those who now believe in the TBM principle but have an unrealistic value of its limitations.

There are some limitations or restraints, and some of these are being erased or modified. Machines cannot be built for tunnels smaller than 80 in. today because they fill the hole too much for maintenance and roof support. TBM's are uneconomical in most tunnels larger than 30 ft in diameter because cost per cubic yard for conventional excavation decreases with diameter increase at a faster rate than it does with TBM's. No one really knows the exact effect of size on the cost or machine requirements for TBM's.

The Jarva Manufacturing Company proved that machines could compete commercially and bore rock stronger than 20,000 psi in St. Louis limestone, and Calweld Division is ready to try its machine in rock stronger than 30,000 psi. There were rules of thumb that a machine should not be considered for tunnels shorter than 2 miles in length because the high capital cost (which is about double that for conventional tunnel driving) could not be written off in less. With the greater availability of used machines, this restraint should not be applied automatically. The availability of used machines is helping to overcome the disadvantage of long lead time of about 10 months to build a tunnel-driving machine.

Much has been written about the advantages of TBM's. Where they can be used, there is good evidence that steel primary support can be cut almost in half. Because of elimination of overbreak, concrete for permanent lining often is reduced by 50 percent. These 2 savings to the owner and contractor in many cases will pay for most of the depreciation on the TBM. There are about 10 percent fewer lost-time accidents in TBM jobs than in conventional jobs; eventually this will be reflected in insurance savings.

Total labor savings for TBM to date have been less than was anticipated. The crew at the heading has been decreased by about 75 percent, but larger crews are required to lay track and handle the muck production at the higher rates. The net saving has been about 15 percent.

Predictions of machines in 20 or 30 years, of course, are impossible to make with any assurance of accuracy. It is a good exercise though if not taken too seriously. Some conclusion may be reached by evaluating announced research plans. The following estimates are made with these reservations in mind. It is hoped that they will stimulate manufacturers, contractors, or tunnel designers to similar thoughts that may help the progress of this technology in which there is such a large stake.

In tunnel drivers of tomorrow, "tomorrow" must be defined. If tomorrow is the next 2 decades, then the tomorrow TBM will be a greatly improved version of today's machine. If tomorrow is the year 2000, then the TBM will be one that destroys rock by a combination of mechanical, thermal, or more than likely high-pressure water erosion.

The 1990 machine will cut rock with rolling cutters. A convex or concave head will cut more of the rock in its weaker tensile mode, rather than compression as is done now. It will be able to set roof supports automatically, and this may be a spray-on concrete or plastic. The support may consist of ribbons of steel that are flexible in storage but are formed and applied in place. It will be completely dust free. It will sense bad rock or water trouble ahead. It will be operated remotely so that men are rarely exposed to unsupported roof. Most of the controls will be handled by a computer responding to a laser beam for guidance and other electronic devices for varying thrust, varying rpm, and avoiding obstacles. The permanent lining will be installed within a few feet of the rear of the machine.

Devices will be available to ream the round or circular openings, made by a machine, to moderately large rectangular sections, as are required for underground urban parking lots and rapid transit stations.

Solids pipelines to handle muck will not replace wheeled vehicles that will continue to be needed for supplies and men. Rail or vehicles or both are cheaper than belts and

do not require crushing or the separating of cuttings from a slurry as is needed in a pipeline. Tunnel cars of 1990 will not look much like those of 1970. They will be long, slender, and flexible, and therefore adaptable to various shapes and sizes of tunnels. They may ride on pneumatic tires on a special prefabricated roadway at double or triple today's tunnel rail speed, which is 10 mph. Each may contain its own propulsion unit and operate manned or unmanned. The 1990 machine will be able to turn curves of a radius equal to 5 times the tunnel diameter as opposed to about 20 times the diameter limit of today's machine. It will be able to go down grades of 20 deg as opposed to today's of about 10.

The 3,000 ft per month or better progress rate of today's machines will be a low average production rate in 1990. Today's infrequent high record rates exceeding 6,000 ft per month will be achieved frequently in 1990, and 200 to 300 ft-days will be common in good ground.

The mechanical parts of 1990 machines will weigh about 60 percent of that of today's, which in tons is approximately 0.6 times diameter in feet squared; but weight of electronic controls, dust controls, temperature controls, and automatic roof support will offset the weight saving. Freight to the job will be about what it is today. Boring-machine business will have developed sufficient volume and highway- and water-tunnel designers will have standardized so that today's lead time for a machine will be 3 to 4 months rather than 10 to 12 as it is in 1970.

Tunnel-boring machines in 1990 still will be unable to penetrate heavy, broken, hard rock ground. Drilling and blasting will remain the standard method for such ground as well as hard rock tunnels larger than 35 ft in cross section.

The machine of 1990 will have cutters that will penetrate hard rock (35,000 psi) at 6 ft/hour at a cutter cost of less than $4/cu yd and that will penetrate rock weaker than 20,000 psi at 25 ft/hour and a cutter cost of less than 50 ¢/cu yd.

Time between cutter changes will be extended from its current 100-hr average (good performance) to 300 rotating hours. Machine reliability and backup equipment will have improved so that machine availability will increase from its present 60 to 85 percent. Cutters therefore will be replaced about twice a month. Part of the reduction in cutter cost will come from cost savings in mass production, reducing the list price, so that it will cost 20 percent less than the approximately $40,000 to $70,000 (depending on rock type) to dress or add a complete set of cutters to a 20-ft machine.

This will make tunnel-boring machines competitive in tunnels in any kind of reasonably competent rock to diameters of 35 ft. There will be a rock TBM for tunnels of 60 in., but not smaller.

A concentrated research effort could produce the 1990 machine by 1980 and move the year 2,000 TBM to 1990.

Some very rough guidelines for estimating machine-bored tunnels are given in Table 1.

TABLE 1

RULE-OF-THUMB GUIDELINES FOR ROUGH ESTIMATIONS OF MACHINE TUNNEL BORING

Item	Unit	Value 1970	1990 to 1970 Ratio
Horsepower	hp	50D	0.7
Machine weight	lb	$D^2 \times 10^3$	0.8
Minimum turning radius	ft	20D	0.25
Rotary speed	rpm	80/D	1
Thrust	lb	$5D \times 10^4$	0.6
Maximum penetration rate	ft/hr	$2/S \times 10^{-5} - 3 \leqslant 25$	1.5
Production	ft/shift	4 × max. pen. rate	1.5
Cutting cost/cu yd	$	$0.50 + (S \times 10^{-4})^2$	0.6
Machine cost	$	$5D \times 10^4$	1.2

Note: D = diameter in ft, and S = rock compressive strength in lb/sq in.

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Design of Tunnel Support Systems

D. U. DEERE, R. B. PECK, and H. W. PARKER, Department of Civil Engineering, University of Illinois;
J. E. MONSEES, A. A. Mathews, Inc., Construction Engineering, Arcadia, California; and
B. SCHMIDT, James P. Collins and Associates, Cambridge, Massachusetts

This paper deals with the results of a recent evaluation of procedures for the design of tunnel liner systems; the relationships among the geologic materials to be tunneled, construction methods, and support systems; practical guidelines for the design of tunnel supports in both soil and rock; and problems associated with improving existing support systems for high-speed tunneling.

•SELECTION AND DESIGN of the support system are only two of many interrelated factors in the overall design of a serviceable and economical tunnel. The type of support, the method of excavation, and the character of the ground are inseparable considerations. If the route is laid out to encounter the worst rather than the best geological features, or if the construction method is ill-suited to the geology, no amount of refinement of the lining can appreciably influence the economy of the job. Nevertheless, for each tunnel layout and each construction method, some types of lining are preferable to others. Initial support during construction and final support during the functional life of the tunnel pose separate requirements; sometimes both are best satisfied by a single support system.

Rational design presupposes a knowledge of the demands on support systems, criteria for successful performance, familiarity with the capabilities of available systems, and methods of analysis verified by experience. Improved practice in the future is likely to have its roots in a clear understanding of the shortcomings and requirements of today's practices. This paper summarizes several current studies on the various aspects of design of the support systems for transportation tunnels.

TYPES AND FUNCTIONS OF TUNNEL SUPPORT SYSTEMS

The basic functions of a tunnel support system are to keep the tunnel stable and to make the opening usable. The specific purposes of support systems, however, depend greatly on the purposes of the tunnel.

Traditionally, tunnel supports have been classified into two groups, temporary and permanent. In modern transportation tunnels, however, no such clear distinction can be drawn. Modern supports do not rot away and thus are not as temporary as the timber sets used years ago.

The first supports installed will probably carry all the loads ever expected on the tunnel as long as the supports do not deteriorate. These supports, which carry either the full load or the greatest share of the load, are called the primary support system. The primary support system must provide the initial support for the opening, control the deformations within the tunnel, and minimize disturbance to adjacent and overlying structures.

Any lining that covers the primary support system is called the secondary liner. In a transportation tunnel, a secondary liner may be required to provide corrosion protection for the primary support system, to provide watertightness, or for environmental

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reasons such as aesthetics. It may be uneconomical and unnecessary to make transportation tunnels watertight because infiltrating water can often be easily controlled and drained from the tunnel. Thus, except for the case of a watertight tunnel, the secondary liner need not be designed as a structural member. In a watertight tunnel, the secondary liner can be designed to share the load with the primary support system.

A savings of up to one-third of the total cost of a tunnel can sometimes be achieved by eliminating the secondary liner altogether (6). On some projects merely making the primary support system corrosion-resistant has permitted elimination of the secondary liner.

TYPES OF PRIMARY SUPPORT SYSTEMS

Rock Tunnels

Three main types of primary support systems are presently used in rock tunnels in the United States. They are rock bolts, steel sets, and shotcrete. Shotcrete is a pneumatically applied large-aggregate concrete. The need for a secondary lining in a tunnel supported by shotcrete depends on the purpose of the tunnel. Table 1 gives present use of the three types of primary support systems for rock tunnels in various rock conditions. Each of the three support systems can be used under a wide range of tunneling conditions, with some limitations in the poorer quality rock.

Recently, the Bernold System has been used with considerable success in poor quality rock in Europe (9). The system consists of the use of pumpcrete to fill the annulus between curved expanded metal sheets that are placed close to the face. Movable steel sets provide temporary support until the concrete cures.

Soil Tunnels

Table 2 summarizes the applicability of several types of support systems in various soil conditions. In contrast to tunnels in rock, only one or two support systems are likely to be both technically and economically feasible in any given soil condition. Soil tunnels often have secondary liners, but shield-driven tunnels have traditionally been constructed without a secondary liner because the primary support system was of cast iron and corrosion-resistant. More modern shield tunnels, lined with concrete or coated steel segments, are also corrosion-resistant and require no secondary liner.

PLANNING AND DESIGN OF TUNNEL SUPPORT SYSTEMS

Planning and design decisions are of two classes, conceptual and detailed. Decisions of the first class are based on considerations of such factors as the purpose of the project; the depth, alignment, and geometry of the opening; the external environment; and

TABLE 1

USE OF PRIMARY SUPPORT SYSTEMS FOR ROCK TUNNELS

Support System	Quality of Rock: Good	Fair	Poor	Very Poor	Remarks
Rock bolts	Yes	Yes	?	No	Difficult or impossible to obtain anchorage in poor and very poor rock.
Shotcrete	Yes	Yes	Yes	?	May not require secondary liner for corrosion protection. Future developments are promising. Supplementary support is required in poorer quality rock.
Steel sets	Yes	Yes	Yes	Yes	Usually more expensive but sometimes is the only system that can be used.

Note: Table reflects 1969 technology.

TABLE 2

PRIMARY SUPPORT SYSTEMS FOR SOIL TUNNELS

Type of System	Remarks
Bolted steel segments	Generally used in poor soil conditions. Too expensive in other soil conditions. Have been coated with corrosion-resistant film and used without a secondary liner (10).
Bolted cast iron segments	Often used for shield-driver tunnels in soft soil. Too expensive in other soil conditions. Does not require secondary liner for corrosion protection.
Bolted concrete segments	Not yet used in the United States. Applicable to poor soil conditions. Does not require secondary liner for corrosion protection.
Unbolted concrete segments	Used only in soil having long stand-up time, such as very stiff clay. Does not require secondary liner for corrosion protection.
Steel ribs and wood lagging or with liner plates	Versatile under most soil conditions except running or flowing sand and squeezing clay.
Liner plates without steel ribs	Used only for small-diameter tunnels.
Shotcrete	Useful in soils having sufficient stand-up time. Cannot withstand thrust from shield. Does not require secondary liner.
Cast-in-place concrete	Used only for small-diameter tunnels in good soil conditions.

Note: Table reflects 1969 technology.

the required watertightness. The results of these decisions constitute the conceptual design of the underground opening. It may include several alternatives.

The detailed design is then performed to provide several alternate construction methods and support systems that meet the requirements of the conceptual design. The tunneling scheme that results in the lowest total cost for the project is selected.

Few decisions in the design process can be made completely independently of each other. The geology associated with alternate axes at different depths and alignments should be a fundamental consideration in the conceptual design. The selection of the depth and alignment determines the geologic materials that must be tunneled. The materials encountered, in turn, dictate which types of construction methods are feasible. Other construction methods, even though intrinsically cheaper, no longer can be considered. The support system must be compatible with the geology and the construction method. Hence, with the geology and construction method fixed, only a few support systems can be considered.

The selection of the route alignment and grade is one of the most important decisions to be made. If unfavorable conditions will be encountered, the resulting high construction costs cannot be offset by refinements in the design of the support system.

The design of a support system is usually a matter of selection. The selection is more complex than indicated by Tables 1 and 2. Throughout planning and design, the engineer needs to be aware that the geology of the material to be tunneled is the most important variable in establishing the design, construction, and, ultimately, the cost of the tunnel.

MODERN CONCEPTS OF THE DESIGN OF TUNNEL SUPPORT SYSTEMS

During excavation, most of the existing stresses in the ground are redistributed around the opening by mobilization of the strength of the soil or rock. The redistribution is often described as arching. Usually only enough support must be added within a short time after excavation to help the soil or rock hold itself up.

Current soil and rock mechanics practice is to recognize and treat the behavior of any system as a complex function of the interaction of the behavior of the individual components of the system. In contrast, previous concepts and theories for the design of tunnel supports have been based solely on assumed loading diagrams; hence, they are unsatisfactory. Furthermore, because the soil or rock being tunneled does not meet the appropriate assumptions, elastic and elastic-plastic theories are rarely satisfactory for predicting the loads in tunnel supports. The designer must somehow account for the deformation in both the soil or rock and the support. The best way to

visualize this interaction phenomenon is by the simplified ground reaction curve shown in Figure 1.

A schematic load-deformation diagram is shown in Figure 1. The ordinate represents the load in a support when deformation of the tunnel walls has ceased. As the soil or rock deforms toward the tunnel, more strength of the medium is mobilized and more stress is redistributed around the opening. The ground reaction curve qualitatively reflects this redistribution. For any given radial deformation, the ordinate of the ground reaction curve represents the load that must be applied to the walls of the opening to prevent any further deformation.

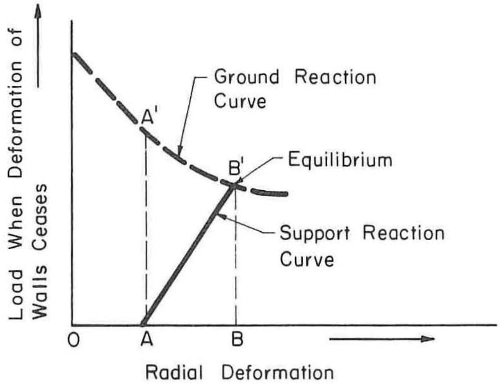

Figure 1. Simplified ground reaction curve.

The inevitable deformation that occurs before the supports can be installed is denoted by line OA. If at this stage a perfectly incompressible support is installed, the load in the support is represented by the ordinate of the ground reaction curve, line AA′, at that deformation. But supports are, in fact, not incompressible. The stress-strain curve of the support is represented by the support reaction curve. While the supports deform radially, the walls of the tunnel also deform until equilibrium is reached at a deformation of the walls of the tunnel equal to OB, a deformation of the supports equal to AB, and a load in the supports equal to BB′.

Unfortunately, at the present time the ground reaction curve cannot be theoretically defined in most materials. Furthermore, even if theory could be used to predict the curve, the large local variations in construction procedures would inhibit the usefulness of the curve for practical design of supports. Research and field instrumentation are continuing to develop these concepts, but for the present the semi-empirical methods described in the following sections appear to be best for practical design of tunnel supports.

GUIDELINES FOR THE SELECTION OF PRIMARY SUPPORT SYSTEMS FOR ROCK TUNNELS

This section presents practical guidelines for the selection and sizing of primary support systems for tunnels in rock. The recommendations are keyed to rock conditions that are described and quantified by a weighted or modified core recovery, RQD (rock quality designation). The RQD differs from the percent core recovery in that the RQD considers only the aggregate length of the pieces of NX core that are 4 in. in length or longer. Shorter lengths of core are not considered. The system is described by Deere et al. (1) and is correlated with the behavior of tunnels by Deere, Merritt, and Coon (2). The rock quality classification is given in Table 3.

TABLE 3

ROCK QUALITY CLASSIFICATION

Rock Quality	RQD (percent)	Approximate Tunnelman's Classification
Excellent	90-100	Intact
Good	75-90	Massive, moderately jointed
Fair	50-75	Blocky and seamy
Poor	25-50	Shattered, very blocky and seamy
Very poor	0-25	Crushed

Guidelines for selection of support systems for 20-ft to 40-ft diameter tunnels in rock are given in Table 4. The table is based on experience and the results of field measurements. The recommended rock load for steel sets is smaller than the upper bound of the original recommendations by Terzaghi (7). Support requirements are reduced in machine tunnels because the rock is not disturbed by blasting. A discussion of the use and

TABLE 4

GUIDELINES FOR SELECTION OF PRIMARY SUPPORT FOR 20-FT TO 40-FT TUNNELS IN ROCK

Rock Quality	Construction Method	Alternative Support Systems: Steel Sets — Rock Load (B = Tunnel Width)	Steel Sets — Weight of Sets	Steel Sets — Spacing[c]	Rock Bolts[a] (Conditional use in poor and very poor rock) — Spacing of Pattern Bolts	Rock Bolts[a] — Additional Requirements and Anchorage Limitations[a]	Shotcrete[b] (Conditional use in poor and very poor rock) — Total Thickness — Crown	Shotcrete[b] — Total Thickness — Sides	Shotcrete[b] — Additional Support[b]
Excellent[d] RQD > 90	Boring machine	(0.0 to 0.2)B	Light	None to occasional	None to occasional	Rare	None to occasional local application	None	None
	Drilling and blasting	(0.0 to 0.3)B	Light	None to occasional	None to occasional	Rare	None to occasional local application 2 to 3 in.	None	None
Good[d] RQD = 75 to 90	Boring machine	(0.0 to 0.4)B	Light	Occasional to 5 to 6 ft	Occasional to 5 to 6 ft	Occasional mesh and straps	Local application 2 to 3 in.	None	None
	Drilling and blasting	(0.3 to 0.6)B	Light	5 to 6 ft	5 to 6 ft	Occasional mesh or straps	Local application 2 to 3 in.	None	None
Fair RQD = 50 to 75	Boring machine	(0.4 to 1.0)B	Light to medium	5 to 6 ft	4 to 6 ft	Mesh and straps as required	2 to 4 in.	None	Provide for rock bolts
	Drilling and blasting	(0.6 to 1.3)B	Light to medium	4 to 5 ft	3 to 5 ft	Mesh and straps as required	4 in. or more	4 in. or more	Provide for rock bolts
Poor RQD = 25 to 50	Boring machine	(1.0 to 1.6)B	Medium circular	3 to 4 ft	3 to 5 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	4 to 6 in.	4 to 6 in.	Rock bolts as required (~4-6 ft cc.)
	Drilling and blasting	(1.3 to 2.0)B	Medium to heavy circular	2 to 4 ft	2 to 4 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	6 in. or more	6 in. or more	Rock bolts as required (~4-6 ft cc.)
Very poor RQD < 25 (Excluding squeezing and swelling ground)	Boring machine	(1.6 to 2.2)B	Medium to heavy circular	2 ft	2 to 4 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Medium sets as required
	Drilling and blasting	(2.0 to 2.8)B	Heavy circular	2 ft	3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Medium to heavy sets as required
Very poor, squeezing or swelling ground	Both methods	up to 250 ft	Very heavy circular	2 ft	2 to 3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Heavy sets as required

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines particularly in the poorer quality rock. See Deere et al. (3) for discussion of use and limitations of the guidelines for specific situations.

[a]Bolt diameter = 1 in., length = 1/3 to 1/4 tunnel width. It may be difficult or impossible to obtain anchorage with mechanically anchored rock bolts in poor and very poor rock. Grouted anchors may also be unsatisfactory in very wet tunnels.

[b]Because shotcrete experience is limited, only general guidelines are given for support in the poorer quality rock.

[c]Lagging requirements for steel sets will usually be minimal in excellent rock and will range from up to 25 percent in good rock to 100 percent in very poor rock.

[d]In good and excellent quality rock, the support requirement will in general be minimal but will be dependent on joint geometry, tunnel diameter, and relative orientations of joints and tunnel.

limitations of these guidelines for specific situations has been published (3). These guidelines, coupled with the designer's personal experience, form a basis for design, although small changes will doubtless be required during construction to account for the inevitable uncertainties.

GUIDELINES FOR SELECTION OF PRIMARY SUPPORT SYSTEMS FOR SOIL TUNNELS

Theoretical studies and full-scale field observations lead to the conclusion that a semi-empirical design procedure is warranted for soil tunnels (3, 5). The procedure consists of four separate steps:

1. Provide adequately for the ring load to be expected;
2. Provide for the anticipated distortions due to bending;
3. Give adequate consideration to the possibility of buckling; and
4. Make allowance for any significant external conditions not included in 1 to 3 above.

For each of the steps, recommendations are given to the extent justified by the present state of the art. Lack of enough information to permit a recommendation indicates a need for further observational data.

Ring Load

The ring load in the lining of a single tunnel, except possibly in swelling clays, is likely always to be considerably smaller than that corresponding to the overburden pressure. Nevertheless, it is suggested that the ring load for design be taken as that due to an all-around pressure γz where γ is the total unit weight of the soil and z is the depth to the axis of the tunnel. Present knowledge is inadequate to permit a more refined estimate. Furthermore, for linings of such commonly used materials as steel, cast iron, or structural concrete, design for a ring thrust to withstand an all-around pressure γz would not usually increase the minimum cross sections that would be used for practical constructional reasons. The design pressure γz also provides a satisfactory allowance for the influence of adjacent tunnels.

Bending

For a single tunnel, an estimate should be made of the magnitude of the change in diameter most likely to occur if a perfectly flexible lining of the same shape as the tunnel were installed in soil comparable to that at the site. A procedure for estimating this distortion is suggested by Peck (5). Field data show that almost irrespective of the rigidity of the lining, and even in soft clays and silts, the change in diameter of a lining rarely exceeds 0.5 percent. If the change in diameter is acceptable with respect to the non-structural requirements, two courses of action are open: (a) to provide an essentially flexible lining such as one consisting of articulated blocks, or (b) to provide a continuous lining that can change shape from circular to elliptical, by an amount corresponding to the change in diameter, without overstress. The limiting stress, whether in the elastic or inelastic range, should be ascertained by the designer according to the stress-strain properties of the material. The second alternative is slightly conservative, because the distortion will be reduced by whatever stiffness the lining possesses.

If multiple tunnels are to be constructed, the same procedure should be followed except that the lining must accommodate the additional distortion associated with the subsequent tunnels. If primary and secondary linings are used, the possibility should be investigated of delaying placement of the secondary lining until all tunnels have been driven.

Buckling

Buckling has been noted in tunnels where supports twisted or were irregularly blocked. However, there is no report of a failure by buckling of a tunnel lining due to earth pressures acting in planes at right angles to the axis of the tunnel if soil or grout

was everywhere in contact with the lining. Provisions should be specified and enforced for uniform closely spaced blocking, uniform filling of the annular space behind shields, or systematic expansion of the lining against the soil. Structural features explicitly designed to prevent buckling can safely be omitted with the exceptions previously mentioned.

External Conditions

The lining should be designed with ample reserve strength for shield-jacking loads and for unsymmetrical or three-dimensional distortions likely at the heading itself. These requirements often govern the thickness of the lining. Reasonable circumferential and longitudinal strength and continuity of semi-rigid linings should be provided to allow for normal adjacent operations such as pile driving or excavating on a small scale.

IMPROVING SUPPORTS SYSTEMS FOR HIGH-SPEED TUNNELING

The future of high-speed tunneling promises many exciting changes in support systems. Innovations are likely to fall into two broad categories: (a) improvements in materials or installation techniques for existing support systems and (b) radically different methods of support. Additional requirements will be imposed on support systems if methods such as the flame-jet or laser beam are used for excavation. If any of these novel methods of rock breakage are successful in attaining production status, support systems will have to be developed that are compatible with the radically different construction method.

If high rates of advance are achieved by using conventional boring machines, the corresponding support systems will have to be both inexpensive and capable of rapid installation. Satisfying both of these requirements concurrently may prove to be difficult. Willis and Stone (8) conclude that from 1970 to 1985, liner installation is likely to represent the constraining factor on the rate of advance of soil tunnels. Mathews (4) discusses several other future problems in the development of support systems. The potential for progress in developing support systems lies in field observations to determine the behavior of actual tunnels during construction as well as in research on innovations in the installation of support systems. Support systems can thus be developed concurrently with the improvements in excavation techniques.

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Tunnel Site Investigations and Geologic Factors Affecting Mechanical Excavation Methods

WILLIAM H. IRWIN (retired), MALCOLM H. LOGAN, and IRA E. KLEIN,
U.S. Department of the Interior, Bureau of Reclamation, Denver

Major efforts are being made by the Bureau of Reclamation to improve investigation techniques, design of underground structures, and construction procedures associated with the rapidly growing mechanical methods of tunnel excavation. The impact of rapid excavation on engineering geologic investigations and the major requirements of the Bureau of Reclamation now being evolved are discussed. Principal attention is given to efforts to improve engineering geologic investigation techniques to meet the demands of mechanical tunnel excavation by development in areas such as geophysics, computer storage and analysis of geologic data, research on drillability indexing, and aerial remote sensing. The paper briefly discusses the application of geologic data involving the development of quantitative values, the apportioning of these values, and their projection to define successive reaches of rock quality and geologic uniformity at tunnel grade.

• AS MAN PROBES the unknown realm of space, he is also, in a less dramatic manner, reaching deeper into the unknown realm of the underground. The importance of this underground work in numerous fields of endeavor has become progressively more important in the past few years. The Bureau of Reclamation, for example, has plans that include several hundred miles of underground water conveyance systems associated with the construction of hydro projects in the western states. These encompass almost every geologic condition that can be conceived.

Major efforts are being made not only to improve the design but also to adapt to rapidly improving construction procedures. Determination of continuity, soundness, and physical properties of the rock to be penetrated is a major essential. The Bureau is also expending considerable effort to improve the exploration techniques and methods of interpretation and evaluation. This is necessary not only to produce a sound, economical project but also to minimize changes that might be required because of unexpected conditions revealed during excavation and construction.

This paper discusses the state of the art of tunnel investigations as practiced and planned by the Bureau of Reclamation; it does not attempt to present standardized proposals for tunnel line investigations relating to mechanical boring.

ENGINEERING GEOLOGY

Engineering Geology and Machine Tunneling

Machine or "mole" tunneling has generated a strong impetus to the expansion of almost every aspect of geologic investigation. There is greater need for more detailed

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information in order to make decisions on the choice between machine and conventional methods. Making this basic decision requires information for evaluating the capabilities of the numerous machines available, for modifying existing machines, or for developing entirely new machines that can cope with projected conditions. It is doubtful whether the tunnel design engineer or the prospective tunneling bidder can have too much information. The point of diminishing returns for investment in geologic investigations is much higher if machine tunneling is in the picture.

Several years ago mole capabilities were limited and could realistically be considered for use only in soft, firm rocks. Now, because of the enormous progress in the ability of tunneling machines to handle hard rock economically, the outlook is entirely different. Practically all proposed tunnels must now be considered for machine as well as for conventional methods. However, adequate care must be given to ensure a balanced, careful investigation, for certainly there are some extremely difficult geologic conditions that will put a machine at a disadvantage or even in an impossible position.

For example, in conventional tunneling the drill-blast-muck cycle may be slowed down or even stopped by hard rock, high groundwater flows, or support requirements. Intermittent progress can still be made by resorting to the use of more dynamite, bigger pumps, or heavier support. There is room to work, and the capital investment in drills and muckers is relatively low.

On the other hand in machine tunneling, if those conditions are encountered, idled capital investment is very high. The lack of room to work may make the obstacles insurmountable and force abandonment of the machine, at least temporarily. If a major change of excavation method is required, extensive extra costs are unavoidable.

The possibilities of combined use of machine and conventional methods on an individual tunnel must always be considered. Blanco Tunnel of the U. S. Bureau of Reclamation's San Juan-Chama Project is an example of a successfully combined operation.

At the present time, the Bureau is studying another project as a possibility for combined tunnel excavation. In this case, a twin bore will be driven parallel to and only 400 ft from one excavated by conventional methods in 1950 with no significant problems. Rock at tunnel grade consists of a series of basaltic lava flows and associated interflow sediments. The basaltic portion of the tunnel will be in rock that varies structurally and texturally from massive nonvesicular to highly vesicular and flow-breccia types. Soft relatively uncemented interflow sediments comprise the remainder of the geologic sequence. Groundwater conditions in the area of the new tunnel have been changed as a result of the operation of the existing tunnel, and saturation of the interflow sediments is now prevalent.

To judge the applicability of a mole, an appreciation of the proportions of the different types of basalt as well as the groundwater and interflow sediments was necessary. Consequently, a core-drilling program substantially larger than the one accomplished for the earlier parallel tunnel was carried out. How the combination of very low-dipping lava flows and wet interflow sediments will be handled as a construction problem remains to be seen, but it is difficult to conceive of an existing tunneling machine that can effectively and economically handle this problem.

Although testing of samples is an integral part of tunnel studies, the amenability of a hard rock mass to either machine or conventional methods is clearly more than a matter of the intrinsic strength characteristics of the rock material as indicated by sclerometer hardness, abrasiveness, compressive strength, elastic modulus, and special boreability index testing. The discontinuities in the rock mass such as joints, bedding planes, and metamorphic cleavage and shears also have a major influence. Therefore, a partial picture (which can be seriously misleading) can be obtained by unbalanced or incomplete selection of samples and application of the resulting laboratory test data. Equal importance must be given to evaluation of systematic descriptions of field geologic discontinuities and the core breakage characteristics.

Even if the value of tests of samples is accepted, there is also a tendency by some to overemphasize the advantages of making laboratory tests on cores directly from the "tunnel elevation" in holes directly on the tunnel line. Except for problems encountered in negotiation of contractor claims, there is often little or no advantage. With a proper understanding of the geologic structures and materials to be traversed by the tunnel,

one can generally obtain samples that are entirely satisfactory and representative from suitably located shallow holes that are not "on line." In fact, judicious use of surface outcrop samples collected by a geologist can, under many circumstances, be effective.

Summary Statements

What is new or different about geologic investigations now that the mole, i. e., rapid excavation of tunnels, is ascendant?

1. The scope of investigations required overall has increased greatly. More time, more drilling, more laboratory testing, and more geologic mapping are necessary in reconnaissance, feasibility, and final design stages. Only a few years ago core drilling for feasibility was practically never done; now it is a normal procedure. Previously, in final design, the main emphasis was exploration of the portal cuts; now this is often a secondary consideration.
2. Equal emphasis is now placed on developing information for the bidder and machinery manufacturer as well as for the design engineer.
3. More laboratory testing of core samples is required.
4. In situ testing in bore holes by geophysical and rock mechanics or soils mechanics techniques to evaluate rock properties is being given increasing attention, but this phase of investigations is being approached experimentally; that is, it is still largely a matter of research and development.
5. Adequate geologic data to permit evaluation of support requirements is more critical for machine tunneling than for conventional tunneling.

Investigation Techniques

Major requirements of the geologic investigations for the Bureau of Reclamation presently fall into a sequence that includes the following:

1. Preparation of a program of exploration and laboratory testing that is in balance with the size of the project and the anticipated geologic complexities;
2. Identification and 3-dimensional projection of the various rock units along the tunnel line, which requires detailed knowledge of the stratigraphy in sedimentary rocks and boundary conditions such as flow structure, fracture patterns, and foliation in igneous and metamorphic rocks;
3. Identification, location, projection, and evaluation of secondary structures such as faults, shear and breccia zones, jointing, folding, and unconformities;
4. Evaluation of the potential or calculated risk of encountering adverse groundwater conditions, presence of gas, squeezing ground, abnormally high temperatures, and any other distinctive geologic environments affecting the tunnel bore; and
5. Presentation of these data on clear, concise geologic maps and appropriate cross sections annotated with significant engineering and physical data.

To obtain data on these elements, the Bureau of Reclamation has directed its attention to the development of several engineering geologic aspects pertinent to tunnel excavation by mechanical means. These are briefly discussed in the following paragraphs.

Geophysics

Geophysics is believed to have a good potential as a method for expanding the measurements of many geologic parameters. A research program, now in progress, will evaluate the capability of geophysical techniques to determine, at depth, the attitude of major fault and shear zones and their characteristics with regard to design and construction. Of particular importance in this respect is the effort to provide a basis for judging "moleability" or comparative rock quality by determining the seismic velocity of successive reaches of the tunnel line.

Parallel objectives are (a) developing effective field operation procedures for mountainous terrane and (b) reducing dependence on expensive, deep drill holes and permitting such exploration to be concentrated at strategic points where direct factual exploration data on rock conditions are most beneficial.

Measurement of stress relief or blast damage or both in excavated tunnels by geophysical (seismic) means is an existing capability.

Computer Storage of Geologic Data

The Engineering Geology and Data Processing Divisions have cooperated in developing an initial data storage and retrieval system for geologic information. The main system involves both digital and alphabetical entry of specific geologic data from which subroutine packages may be extracted and analyzed. Separate subroutines have been written for geologic and engineering data such as permeability, joint indexes, fracturing, and degree of weathering, and then provide for storage, retrieval, analysis, and presentation of these data.

The total system promotes organization of these data collection and analyzes, maximizing speed, thoroughness, standardization, and efficiency. In addition it will permit integration of data from numerous projects, both completed and proposed, over long time spans. It stimulates accumulative engineering geologic analyses relating geologic parameters to engineering properties of rock and soil materials in foundations and excavations.

The computer program does not replace the engineering geologist. It provides a modern tool by which he may quickly evaluate a large number of geologic combinations that would be impractical by hand methods of calculation. Thus, for example, searches can be made for combinations of conditions that would produce failure patterns and establish the geologic units in which these circumstances occur.

Joint Studies

A thorough study of the rock joint systems in the terrane through which the tunnel will pass will be of prime importance to the tunnel engineer who must evaluate tunnel excavation techniques and estimate the quantity and suitability of various support methods. In the analysis of the joint data, an attempt is made to answer the following:

1. How many distinguishable joint systems occur in the area?
2. What are the attitudes of these joint systems relative to the proposed tunnel centerline?
3. What is the spacing of the joints within a joint system?
4. What is the typical 2-dimensional extent of joints within a system?
5. What are the widths of the joints in a system?
6. Do joints have a filling, and, if so, what is the filling material?
7. Do joints of one system tend to intersect but not continue across the joints of another system?
8. Has slippage occurred along any joint or system of joints?

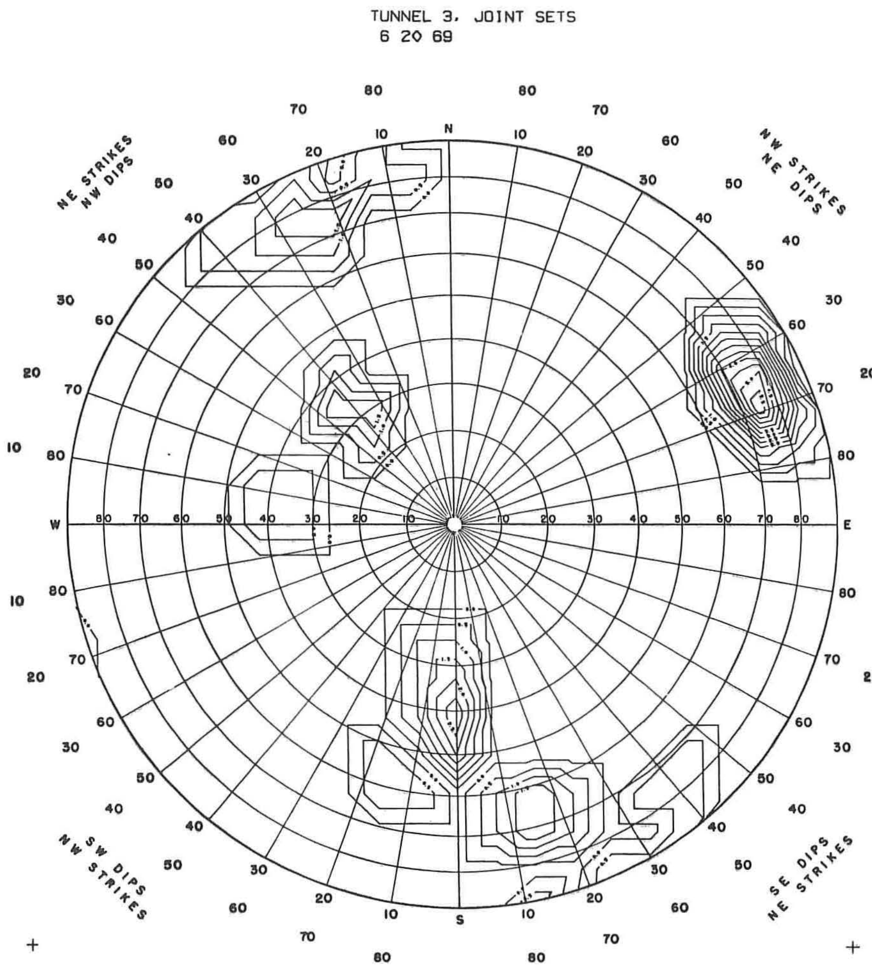
It is quite obvious that even a skeletal evaluation of these data could be extremely difficult. With the services of a computer, successive combinations of different geologic and physical conditions can be analyzed in minutes and, if desirable, presented graphically by automatic machine plotters (Fig. 1).

Drillability Index

During the past year, the Colorado School of Mines has been under contract with the Bureau of Reclamation to accomplish basic studies leading to the development of a drillability index to be used in conjunction with machine tunnel boring projects.

Three basic rock testing procedures have been employed. One consists of a linear cutter, one a button cutter, and the third a single button.

Force-displacement or energy curves for various rock types are being developed. These will be related to standard physical properties and to a commercially available "calibration rock" (Colorado red granite). Interested tunneling contractors can economically obtain samples of the calibration rock. They will have the standardized reclamation test results available for comparison with their own proprietary test techniques and related computations for cutter wear and rate of heading advance. On the limited



Each joint set as recorded in longs of tunnels, drifts, and/or raises is plotted at the equal-area projection of the intersection of its normal with a reference (upper) hemisphere. The value of the point for contouring purposes is 1.0. Points are contoured by summing the value of all points within circles of the 1 percent area on the hemisphere, centered on points that form a grid with a spacing 1/10 of the radius of the reference sphere. Contours represent number of joint sets per 1 percent area. Contour densities indicate the orientation and approximate relative frequence of occurrence of important joint sets. No information on the spacing of the joints within the various sets can be obtained from this drawing. The outer circle of numbers is used for reading strike azimuth and the inner circle of numbers for reading dip azimuth. Numbered circles indicate dip angle.

Figure 1. Contoured joint diagram, equal-area projection, upper hemisphere.

number of samples tested to date, reproducibility of results has been very good. Vertical force response to crushing, cratering, and fracturing have been observed for several reliable samples with close correlation for all data. Numerous test samples have been obtained from reclamation tunnels where the results are being directly compared with the operation and reaction of the tunneling machine.

Instrumentation

The Bureau of Reclamation has, to date, installed multiple position borehole extensometers (MBPX) in 6 underground structures to measure the relationship of rock movement to the adjacent excavation. Installation of the instruments has been both external and internal, that is, from both the ground surface and in the tunnel adjacent to the working face respectively. Present capabilities allow installation of the equipment to a depth of 250 ft from the ground surface (Fig. 2).

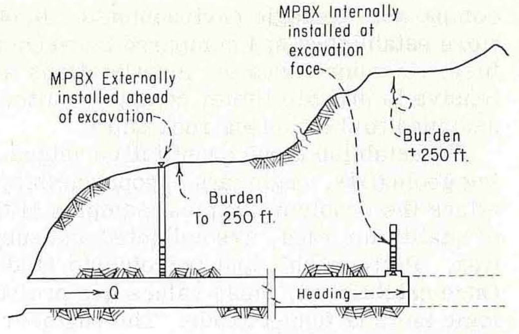


Figure 2. Typical tunnel installation of rock extensometers.

Evaluation of the tests conducted thus far indicate that tunnels excavated by mechanical means (mole) maintain the inherent competence of the surrounding rock and result in less movement than those excavated by conventional drill and blast methods.

Air-Photograph Interpretation—Remote Sensing

For all but very short water-conveyance tunnels, the interpretation of vertical aerial photographs proceeds concurrently with field geologic mapping along the tunnel route. Such photographic studies supplement and are an indispensable aid to developing an understanding of underground conditions. Lineations may be detected in air photographs that are essentially invisible in ground surface field examinations. The lineations may be a guide to the identification of formation boundaries, wide shear or fault zones, and different joint systems that will have a marked influence on evaluation of geologic conditions at tunnel grade.

Distribution of vegetation, anomalous alignment of ravines, stream patterns, and other drainage or linear features may reveal the need for more detailed mapping or assist in locating drill holes. Air-photo patterns are commonly examined for a considerable distance on both sides of the tunnel line for evidence of discontinuous regional lineations. Such lineations may represent important changes in geologic structure or zones of weak rock that are covered in the immediate vicinity of the tunnel line itself.

Evidence is growing that some techniques of aerial remote sensing can reveal features, such as fault lines and anomalous distribution of near-surface groundwater (reflecting subsurface structure), that cannot be detected in aerial black-and-white and standard color photography. Side-scanning radar, infrared scanning imagery, and color infrared currently show the most promise in potential application to tunnel line investigations.

APPLICATION OF GEOLOGIC DATA

The design of engineering works in rock combines empirical practices, proven theoretical analysis, past experience, and an understanding of the major parameters of the rock involved. Principles from mechanics have been used to predict rock behavior, but most of the time serious shortcomings are inherent and impede the geologic and engineering analysis because of the inability to handle anisotropic rock masses. However, mechanical methods of tunneling are believed to be more amenable to analysis than conventional methods because the blast-damage factor is eliminated, the natural strength and architecture of the rock is retained, and the rock mass at tunnel grade can be more validly related to in situ surface conditions along and adjacent to the tunnel alignment.

In order to effectively utilize the geologic information, an attempt must be made to reduce these data to quantitative or engineering terms. This has commonly been accomplished by a relationship to past tunneling experience in comparable or near-

comparable geologic environments. In other instances, the relative estimated values were established and compared between geologic units anticipated for the individual feature. In some cases the investigations (both field and laboratory) were sufficiently extensive to provide rather complete suites of tested samples from which values could be assigned to the various rock units.

To establish these quantitative values along with the distribution of each, engineering geologists, engineers, geophysicists, and laboratory technicians work together to attack the problem. Typical samples of the various rock types, representing a range of quality for each, are collected and subjected to laboratory tests for physical properties. Petrographic and petrofabric studies are also conducted on the test samples. Once established, these values are projected and apportioned to their respective geologic units at tunnel grade. The engineer then utilizes the quantitative information to develop a design.

Although field experience in machine tunneling is far more limited than in conventional tunneling, lining and supports appear to be the major item of design concern to reclamation projects. In the conventional tunnel, discontinuities or weaknesses in the rock structure are generally accentuated by blasting, and immediate attention is given to support requirements, construction practices, and permanent lining requirements as construction proceeds. Conversely, elimination of blast effects in machine tunnels (commonly neatly cylindrical) obscures the defects in the tunnel wall and slows down the reaction of the rock around the tunnel to the new stress conditions. The newly machine-excavated tunnel walls may, therefore, appear stronger than they actually are and may lead to initial misjudgment of support and lining needs. Thus, although this paper is essentially concerned with investigations in advance of actual tunneling excavation, we believe it is worth calling attention to this significant difference between the machine-bored and blast-excavated tunnels. It would appear that the demands on the engineering geologist for thorough geologic mapping during construction and continued observations on the behavior of the rock are even greater for the machine than for the conventional tunnel; that is, geologic investigations must be continued into and beyond the construction period for machine tunnels to an extent beyond that envisaged for conventional tunnels.

Between the time that the face has passed and stability of the excavation has been achieved, it has been found that the vertical height of the tunnel opening has diminished. The actual distance at which stability is achieved varies from tunnel to tunnel and even in the same tunnel. Instances have been noted where stability of the excavation occurred within 2 tunnel diameters of the face. In other instances, stability has been noted to occur about 200 to 300 ft from the face, or in the order of 10 tunnel diameters. If the concrete lining is in place when this adjustment is taking place, bending moments will be developed in the lining; and cracking may occur unless the concrete is still sufficiently plastic. By delaying placement of lining beyond the rock load adjustment period, abnormal stress buildup in the lining due to instability of the rock can be minimized. The design studies, therefore, require that geologic investigations develop information regarding the geologic structure and its potential for failure in order that the design required to prevent such failure may be developed.

In this respect, a major frontier of subsurface geologic investigations for tunnels is the determination of the actual state of stress (in situ stress) of the rock along the line. This problem can be approached as a rock mechanics problem by devising tests to be made in bore holes and from analysis of the geologic history. Work has hardly begun in this challenging field.

Weakly lithified sedimentary rocks, particularly argillaceous ones such as many shales and claystones, require special attention, especially when traversed at great depths. When these rock types prevail along the tunnel line, the use of mechanical tunnel methods have marked advantages over conventional excavation methods.

The natural groundwater conditions and the influence of construction operations on the moisture content and related strength characteristics must be understood. Sampling for laboratory testing and in situ testing by geophysical or other methods and the evaluation of the results are part of the engineering geologists' work in tunnel investigations.

Inasmuch as the circular tunnel cross section is the strongest geometrical configuration for stress distribution, less or lighter support is required to maintain the integrity of the bore. Also, elimination of the blast damage factor allows a minimum support requirement because of the absence of overbreak.

Shale interbeds, strongly developed joint sets trending parallel to the tunnel, faults, and severely altered or sheared zones are examples of geologic structures that exert a marked influence on support requirements. In machine tunneling an early evaluation of support needs is basic to the selection of the mole type.

CONCLUDING STATEMENT

Engineering geology has had a difficult task in meeting the requirements of conventional excavation methods and the advent of the tunneling machines, for rapid excavation has compounded the requirements for more detailed and reliable geologic knowledge of underground excavations. Machine tunneling or rapid excavation is only in its infancy, and the technology associated with it remains as a formidable challenge not only in the Bureau of Reclamation but in numerous other fields of endeavor.

Summary Remarks

WILLIAM N. LUCKE, Office of High Speed Ground Transportation,
U.S. Department of Transportation

•WE ARE indebted to Day for his informative description of present nuclear excavation technology. The potential of this technique for such large projects as the interocean canal staggers the imagination. I can only assume that the development of "clean" nuclear explosives has progressed to a point where radiation effects are no longer a problem.

When one considers the advances that have been made in tunneling machines during the past 10 years, Williamson's projections for the future seem very conservative. On the other hand, I am not optimistic that any really significant advances can be made by simple improvements in the engineering design of cutters and bearings. I think that some innovative approach will be necessary to achieve advances of any magnitude. In particular, I am hopeful that some of the research we in the Department of Transportation are doing will provide a dramatic increase in boring machine capabilities, as well as permit economical operation in the harder rock formations. Specifically, we are exploring the possibility of chemically or thermally weakening rock ahead of the cutter blades. It is well established that chemical treatment or heat will weaken rock strength, but a major question remains as to the practicality of such methods in a tunnel environment and the cost benefits to be realized. We hope to have some answers to this soon.

Standardization of machines is a worthwhile goal. Unfortunately we have no organizational mechanism for deciding on diameters and other factors that might be associated with standardization.

A rather interesting point was made by Irwin et al. in their statement that smooth tunnel walls produced by tunneling machines may hide poor internal conditions and thus provide a false sense of security that might lead to liner design error. This certainly highlights the need for post-excavation geologic exploration.

Another issue raised, which I consider to be rather controversial at this time, is whether lining should be placed immediately after excavation or after stress relaxation has taken place. One might argue that forces on the liner will be less after stress relaxation and therefore could be of lighter design. On the other hand it has been well established that shotcrete, for example, is most effective when placed immediately after excavation so as to assist the development of arching action before degradation of the rock structure occurs.

I also found interesting the comment by Irwin et al. that it is not always necessary or desirable to explore on the "tunnel line" and that outcrops and other evidence, properly evaluated, can result in valid conclusions as to the subsurface geology.

The theme expressed by Irwin et al. that machine tunneling requires more comprehensive exploration is certainly valid, as has been borne out through several experiences in which unexpected bad ground necessitated removal of the boring machine at considerable expense.

Geologic uncertainty is of course reflected as a cost of risk in contractors' bids. The greater the uncertainty is, the higher the cost. On the other hand, geologic exploration also costs money. At some point the law of diminishing returns must govern the economic trade-off, so that some reasonable limit to the extent of geologic exploration can be established. I know of no hard and fixed rules to the game, and each case must certainly be viewed on its own merits.

Paper sponsored by Committee on Soil and Rock Properties and presented at the 49th Annual Meeting.

I should like to add to the comments by Deere et al., with regard to the need for secondary lining in transportation tunnels. While steel sets, shotcrete, or segments may be quite satisfactory for urban transit systems, the aerodynamic losses that would result from using these types of final liners in advanced high-speed intercity systems would preclude their use. In these latter systems liner surfaces must be smooth and have a low friction coefficient. Since such systems would probably be below the water table, liner systems must also be capable of handling water pressures and leakages.

Shotcrete is a relatively new liner method in this country but is gaining wide acceptance. One disadvantage is that the resulting surface is not smooth or uniform. We see a definite requirement for someone to devise a placement method that incorporates an automatic troweling feature so as to provide a uniform cross section and smooth surface.

I concur with the authors that field observation is needed to determine the behavior of actual tunnels both during and after construction. Much can be accomplished by theoretical analyses of support systems, but empirical data are necessary to confirm and expand our theoretical knowledge. The University of Illinois, under Department of Transportation sponsorship, is now developing a program for instrumenting on-going projects for this purpose. It is hoped that similar programs will be developed by other tunnel construction agencies so as to provide a well-documented storehouse of data on various kinds of tunnel designs and geologic situations.

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